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Framework of the Equitable Building

Steel erection recently began on what will be the largest office-building in existence, the Equitable Building, New York City, on the site of the burned office-building of the Equitable Life Assurance Society.* The large size is the product of an unusual area of ground plan and an unusual (but not unprecedented) height. But it must be remembered that the very tall buildings, as the Singer, Metropolitan, Central Union, L. C. Smith, and Woolworth, are tower buildings, the main buildings being of moderate height and only a narrow section or tower extending up beyond this. The Equitable, on the other hand, represents a reversion to the normal type of building, the ground-plan area being carried up undiminished to the top. In part this may be due to the fact that it occupies an entire block (Broadway to Nassau St., Pine St. to Cedar St.) over which the old Equitable Building spread in its heterogeneous growth.

*For notes on the fire of Jan. 9, 1912, and the old building, see "Engineering News," Jan. 18, 1912, p. 123, and Nov. 28, 1912, p. 986.

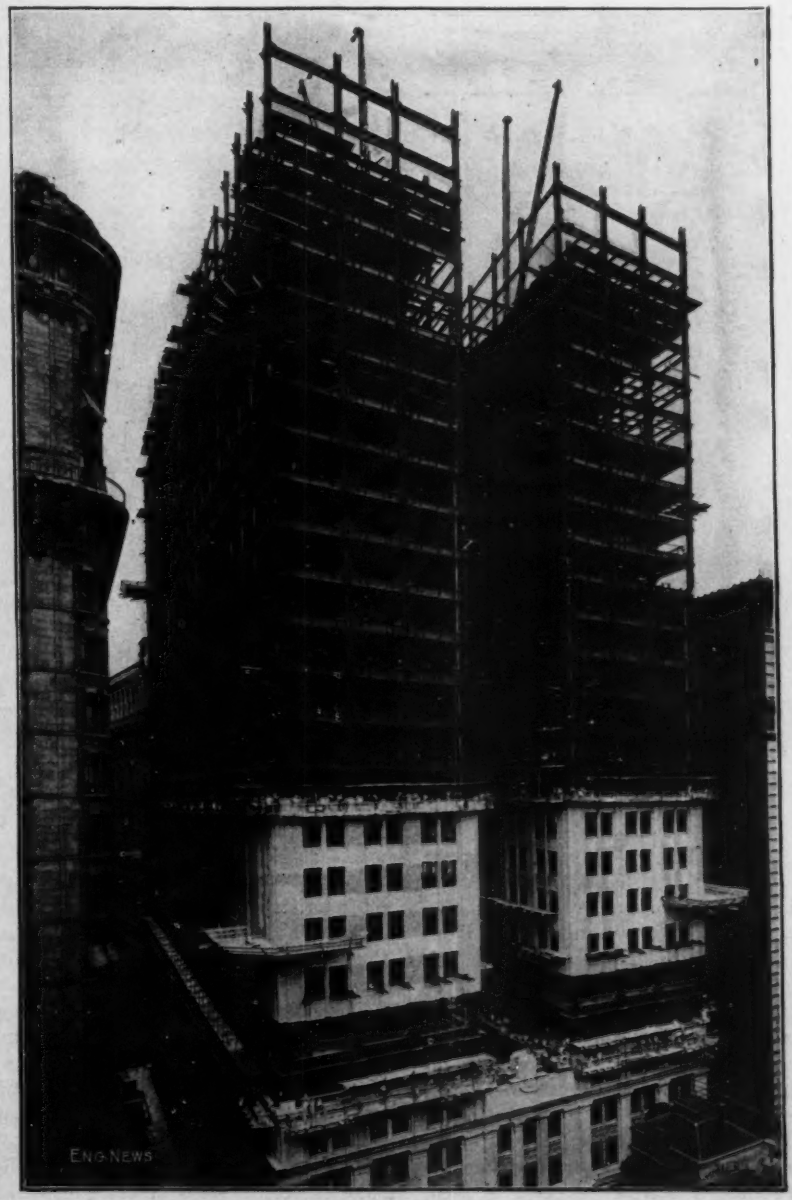


FIG. 1. THE EQUITABLE BUILDING ON JULY 16, 1914

Structurally the building is of remarkably regular design. All its columns run through from footing to roof, and there are no trusses or similar complications—an unusual condition in so large a building. The frame is composed essentially of columns, beams and wind-bracing.

The bank vaults in the basement, as well as engines and boilers, etc., are carried on their own foundations, independent of the steelwork, thus eliminating the weight of framing which would have been required to support them.

The Equitable has three stories below ground and 36 main stories above, but these contain two intermediate floors (3 to 4, 34 to 35), making 41 tiers of beams, inclusive of the roof. The building is about 542 ft. high from Broadway sidewalk to coping; measured from the level of the normal foundation grillages, it is about 600 ft. high.

The area of the block occupied by the building is slightly more than an acre, being 159 ft. in width average and 308 ft. in length average (the outline of the block has no right angles and no parallel lines). Thus the total floor space is nearly 40 acres.

Great Weight of Framework

While ordinary office-building floor loads are figured on in the design of the building, the mere size of the structure makes the weight of of the framework run to unprecedented figures. The following little tabulation is instructive:

WEIGHT OF STEEL FRAMEWORK IN SOME LARGE BUILDINGS

Equitable building, New York	32,000
Wanamaker, Philadelphia	28,000
Woolworth, New York	26,000
Hudson Terminal, New York	20,000
Continental National Bank, Chicago	16,000

There are several buildings in the neighborhood of 20,000 tons of steel. It is seen that the Equitable Building is from 10 to 40% in excess of what has heretofore been achieved in office-building construction, in point of size as gaged by weight of frame.

and this section is developed to areas of over 390 sq. ft., about twice as great an area as ever before used with this form of column.

General Arrangement

The floor plan (see windbracing sheet) shows that the building is arranged in H-form, courts being cut in at the middle of the Nassau St. and Broadway fronts, each 35 ft. wide and about 77 ft. deep; these courts, however, exist only above the sixth floor. The floor area within outer walls is 45,000 sq.ft. per floor below the sixth, and 40,000 sq.ft. above.

The elevators are in six banks of eight each, in the center of the building, extending to the following floors, respectively: 13th, 21st, 28th, 33rd, 36th and 38th. In

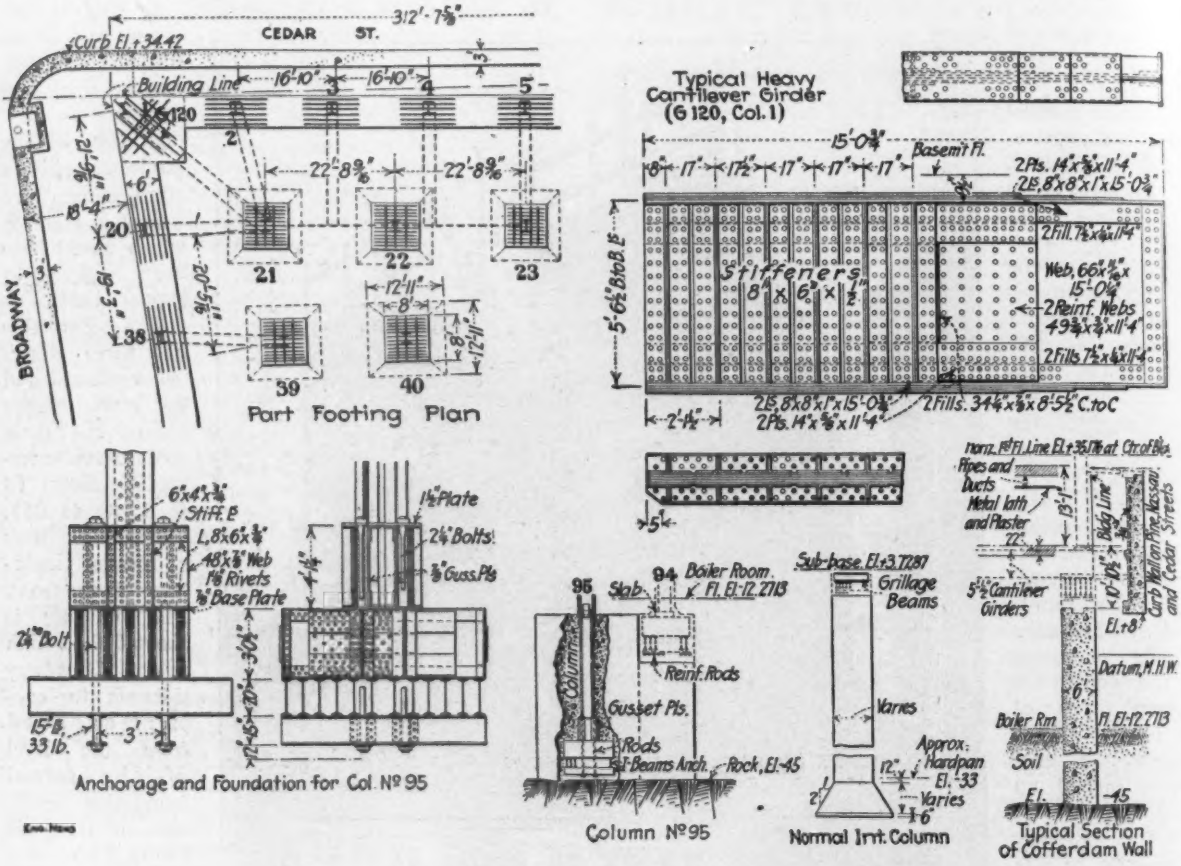


Fig. 2. Typical Footing Sketches, Equitable Building

The windbracing is fairly normal and regular, in comparison with that required in tower buildings. However, it is highly individual in its arrangement, and is correspondingly interesting. As will be noted later, there are eight vertical wind-trusses and one horizontal wind-truss. On account of the number and depth of wind-trusses, the concentration of wind load on columns is less than where the windbracing is necessarily more concentrated. Therefore, while the Equitable has some remarkably heavy column sections, none of them are beyond precedent. Their type, however, is novel for extremely heavy construction; a simple I-section is used, conjunction with a central corridor and service spaces they take up the core of the building, so that all the offices are along the outside; this largely eliminates the need for internal day-lighting.

The floor framing is simple, as suggested by the plan. Beams are spaced 5 to 6 ft. They are set generally with top flanges 4½ in. below finished floor surface, and carry tile-arch floors of 12-, 14- and 16-in. tile. These are end-construction flat arches, with end-construction skewbacks —the latter an unusual feature.

The wall architecture is also sufficiently regular to make the connections of the steelwork at the walls rela-

tively simple. A few spandrel sections are illustrated in Fig. [illegible].

Foundations

In the original design, most of the piers for the interior columns were to be carried only to the hardpan overlying the rock (as shown in Fig. 2). Before the piers were built, however, it was decided to carry them down to rock. All these piers were built in open pits.

The entire exterior line of columns is carried on *pneumatic caissons* sunk to rock, and these caissons are arranged to form a tight coffer-dam wall, retaining the surrounding earth and excluding water. The steelwork of basement and sub-basement floors braces this coffer-dam wall.

Four of the interior column piers are specially heavy, and contain steel anchorages. These are the main piers of the principal central windbracing system (systems *C*, windbracing sheet), being cols. 44, 47, 92 and 95. A general sketch of one of these piers is given in Fig. 2. Alongside is a sketch of one of the normal interior foundations.

The section through the coffer-dam wall and curb retaining wall in Fig. 2 illustrates the foundation arrangements at the street line.

In connection with the foundations it is to be noted that the present practice of the Building Department in New York City prohibits foundations extending beyond the building line into the clearway of the public streets. This means that the outer edges of the grillage footings must be inside the building line and the center of the footing therefore some distance back of the building line. As the column ordinarily will not be so far back of the building line, cantilever footings must be used in order to center the load on the grillage. Thus the entire outer ring of column bays in the building contains cantilever footing girders. At the corners of the building the cantilever girders of the corner column and the two adjoining wall columns were carried back to the same interior column for anchorage. This results in a three-forked girder arrangement at the corners, as shown by sketch in Fig. 2.

At the four main columns of the central windbracing system, the wind stress exceeds the dead-load at top of concrete pier, giving an uplift on the windward side. For this reason the column base is anchored and a special connection of base to column provided to transmit the tension; and the splices in the lower section are proportioned to transmit the tension.

The concrete caps of all foundation piers, under the column-footing grillage, are reinforced by two layers of steel rods in the concrete, although the grillage beams cover the entire area of the pier.

The I-Shaped Columns

Strict uniformity was maintained in the column design, all sections adhering to a set of three types, namely: Simple I-section, consisting of four angles and web; I-section with covers; and I-section with covers and web reinforcing. However, there were three fixed depths of web (13, 14 and 17 in.), and three widths of cover-plate (18, 20 and 24 in.), so that 13 separate series of column sections resulted. The following remarks on the several classes begin with the lightest (upper stories) and go downward:

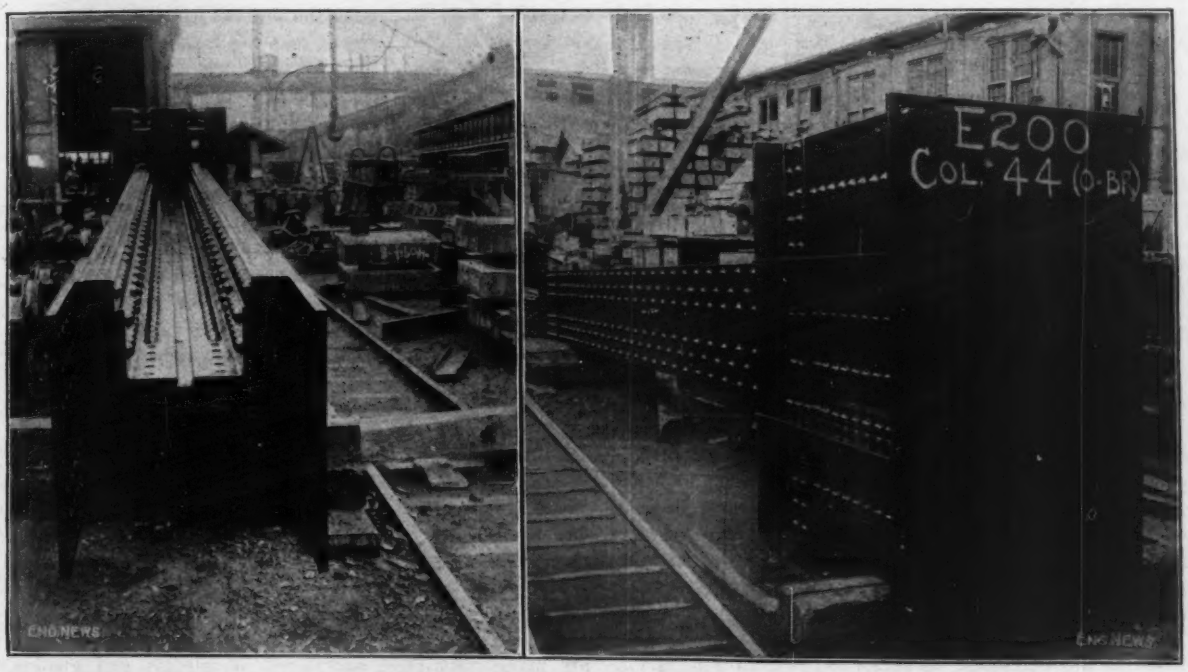


Fig. 3. An Anchor Column of the Equitable Building, in the Pencoyd Shop Yard

(American Bridge Co.)

Simple I-section—Classes A, B and C, of web depth 13 in., 14 in. and 17 in., respectively. Range of make-up, from four angles 5x3x⅝-in. with ¾-in. web, to four angles 8x8x1⅛ in., with 1⅛-in. web. Range of area, 17 to 72 sq.in.

I-section with 18-in. covers—Classes D, E and F. Body, four angles 8x6 or 8x8x⅝, with ¾-in. web; cover-plate thickness per flange, from ⅜ in. to 2 in. Range of area, 69 to 139 sq.in.

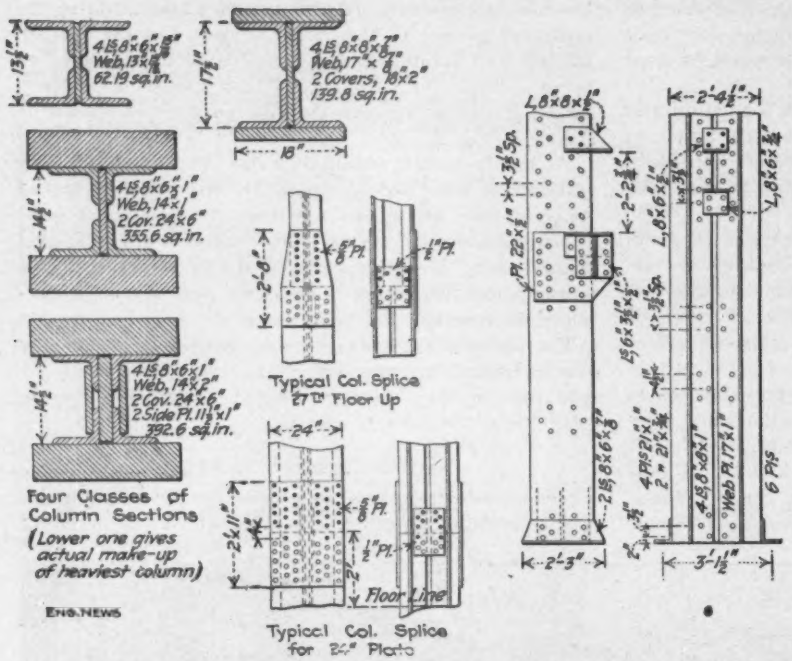

4 L's 8"x6"x 7/8" Web, 13"x 7/16" 62.19 sq.in.
4 L's 8"x6"x 7/8" Web, 17" x 7/16" 2 Covers, 18"x2" 139.8 sq.in.
4 L's 8"x6"x 1" Web, 14"x 1/2" 2 Cov. 24"x 6" 355.6 sq.in.
4 L's 8"x6"x 1" Web, 14"x 2" 2 Cov. 24"x 6" 2 Side Pl. 11 1/2"x 1" 392.6 sq.in.
Four Classes of Column Sections (Lower one gives actual make-up of heaviest column)
Eng. News
Typical Col. Splice 27th Floor Up
Typical Col. Splice for 24" Plate
Floor Line
L 8"x8"x 1"
L 8"x6"x 1"
6 Pls

FIG. 4. TYPES OF COLUMN SECTION, EQUITABLE BUILDING

I-SECTION WITH 24-IN COVERS—Classes J and K (14- and 17-in. web). Body, four angles 8x6x1 in., with 1-in. web; cover-plate thickness per flange, 3½ in. to 6 in. Range of area, 222 sq.in. to 356 sq.in.

I-SECTION WITH SIDE PLATES—Class M (14-in. web). Body, 4x6x1 in., web, 14x1 in. to 14x2 in.; two side plates, 11½x1 in.; cover-plates per flange, 24x6 in. Range of area, 373 to 393 sq.in.

The thicknesses of cover-plate, etc., refer in all cases to total thickness specified. It was left to the contractor to decide upon the number of the plates with which to form this thickness. For the thick covers, 1-in. plates were generally used, partly because more readily obtainable in long lengths, and of more assured quality, than thicker plates.

Four sizes of rivets were used in the columns, as well as elsewhere in the work: ¾-in. rivets for grips of less than 2 in.; ⅞-in. for grips from 2 in. up to 4 in.; 1-in. rivets for grips of 4 in. to 6 in.; and 1⅛-in. rivets for grips of 6 in. and over.

A set of type sections of column and an actual cross-section of the maximum column are given in Fig. 4.

The column of greatest sectional area, 392.6 sq.in., has a length of 41 ft. 3½ in., and weighs 28.8 tons. Its maximum load is 2635 tons. Another column weighs more, however, on account of its greater length (44 ft. 9½ in.). Its weight is 30.8 tons. The longest column section, 47 ft. long, weighs 25.7 tons. The character of the detailing and riveting is shown in Fig. 4 by a copy of the shop drawing for one of the heavy columns, and by Fig. 3, a view of one of the anchor-column sections.

Steel-slab baseplates are interposed between column and grillage.

WINDBRACING

The end areas being small and the resisting sides (east and west) being very long, no special provision is made for windbracing on the long north and south fronts of the building. All spandrel girders on these two sides, however, have top and bottom connections as well as web connections to the columns, and therefore offer ample resistance to pressure in a longitudinal direction.

In the shorter or transverse (north and south) direction, where the total width of the building is about half the length and where the width of the wings alongside the courts is about one-sixth the length, there are no less than eight planes of windbracing (see plan on windbracing sheet). Each plane was designed to support the building halfway to the adjoining plane of bracing on either side.

The planes of bracing are as follows: One each in the Broadway and Nassau St. fronts; one each in the back walls of the courts (systems A and A^1); and four lines between the A-systems, namely systems B, C, C^1 and B^1. The front and A-systems extend the full width of the building, except that the former are divided by the courts which extend above the sixth floor. The B- and C-systems, however, are confined to the three center panels of the width, where they do not infringe on office space. Therefore the windbracing problem in the middle part of the building was just as serious as in a very much narrower building of the same height.

Further, the B- and C-systems are not entirely equivalent, for the C-systems, forming the central bracing, extend to the 32nd floor, while the B-systems go up only to the 23rd floor. Above the 23rd floor the C-system must carry the entire building between back walls of courts, a length of about 150 ft. At the 23rd floor, therefore, where the B-systems start and the A-systems broaden ffom the width of the wings to the full width of the building, a horizontal truss is formed in the floor, tying together systems B and C. At this level the wind shear of the C-systems is assumed to divide equally between the B's and the C's, but, of course, the accumulated column wind stress in the C-systems remains there.

The front-wall system and A-system of wind bracing are made up almost exclusively of girders and gussets or kneebraces. The four interior B and C windbracing systems have girder bracing in the center panel, the outside panels having diagonal bracing. Girders with gussets are used in the middle panel for the reason that this panel must be left unobstructed for corridors (gird-

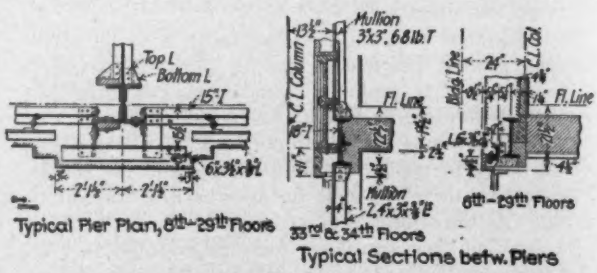


FIG. 5. TYPICAL SPANDREL SECTIONS, EQUITABLE BUILDING

ers and gussets are also used in some special cases in the side panels of the B-system). Up to the 13th floor, double girders and gussets are used, for the reason that with single-girder construction the girders would have been so deep as to infringe on the headroom of the corridors. Single girders and gussets are used from the 14th floor upward.

In the upper floors the diagonal bracing of the B- and C-systems is in panels two stories high. In the three lower stories the diagonal system is one story high, with intermediate horizontal struts run from the point of intersection of the bracing to the point of counterflection of the columns next to the corridors.

The two main fronts, on Broadway and Nassau St., have large entrances two stories high. The high unsupported lengths of the columns at these points are reinforced by auxiliary columns, with diagonal windbracing between the two.

Except in the 23rd floor, no interior horizontal floor bracing is used, the floor construction itself being relied upon to transfer by shear in each floor the wind pressure from panel to panel and finally to the exterior or interior vertical windbracing systems.

Typical details of the windbracing are shown on the windbracing sheet. The proportioning of the diagonals will be obvious. The knee-braced wind girders range generally from 24- to 42-in. depth, with webs generally from $\frac{5}{16}$ to $\frac{1}{2}$ in. thick though some are as thick as $\frac{11}{16}$ in. The flange angles are 6x3½ in. or 6x6 in., in thickness from $\frac{3}{8}$ in. to $\frac{7}{8}$ in. These ranges of dimensions may be taken as increasing quite regularly from the 32nd floor to the bottom, the lowest values applying to the upper five or six stories of windbracing.

Design of Windbracing

The wind pressure was assumed at 30 lb. per sq.ft. over the entire surface of the building from sidewalk to coping, in accordance with the New York building code. The method of analysis of wind stresses is substantially the same as has been followed in a considerable number of large buildings.

The fundamental assumptions were: (1) Points of contraflexure of columns and girders are at mid-span and mid-height, respectively; and (2) column reactions vary as the distance from the neutral axis of the group. The principle of the method, strictly, requires that the column reactions should be proportional to the products of sectional area and distance from neutral axis. But for simplicity it was assumed that all columns are of equal area; that is, the wind reaction attributed to a particular column of a bent depended only on its distance from the neutral axis of the bent and not upon its area as compared with the areas of other column sections in the bent.

The vertical shears in the panels of a bent for a given story accumulate toward the neutral axis. These shears represent the increment of the column reactions of that story; hence the vertical shear in the center panel is equal to all the column reaction increments on one side of the neutral axis.

The horizontal shears in the columns of a bent increase toward the neutral axis, and represent the forces required to balance the action of the vertical shears in the panels acting on any one joint.

To facilitate calculations in a system of more than three columns, the effective depth is first established. This effective depth is equal to the distance between resultants, which are located at the center of gravity of the column reactions on each side of the neutral axis of the system. The value of these resultants is 100%, while the percentage value of any one column varies according to its distance from the neutral axis.

This cantilever method was adopted some years ago by the engineering department of Graham, Burnham & Co., in charge of J. G. Giaver. It is now used in figuring wind stress in all tall buildings designed by this firm. In the earlier period of high-building construction, the engineers commonly considered only horizontal shears produced by the wind pressure.

Detail Procedure—The procedure in applying this method was as follows: Taking a given plane of windbracing, for instance, one side of the Broadway front, the effective depth of this "wind-truss" was computed, i.e., the distance from center line of bent to the line at which the aggregate actual tensions and compressions in the columns would give the same resisting moment. In this case, the bent being 62.6 ft. wide, the effective depth was found to be 52.4 ft. Denoting by 100% the chord stress, which would result in each of the two chords of a truss of this effective depth, the percentages of wind reaction in the several columns of the actual bent were found. These percentages, being constant at all floors, hold good for the entire bent, so long as its dimensions did not change.

Second, the wind shear generated at each floor level of the bent was computed, and by the successive addition of these shear increments, the total wind shear in each story was found. These shears, multiplied by the distance center to center of successive stories, gave the overturning moments in the several stories, which must be resisted by the column stresses. Dividing this moment by the effective depth of wind truss (as previously found) yielded the 100% chord-stress figure, which, upon multiplication by the percentages belonging to the different columns of the bent, yielded directly the column wind reactions.

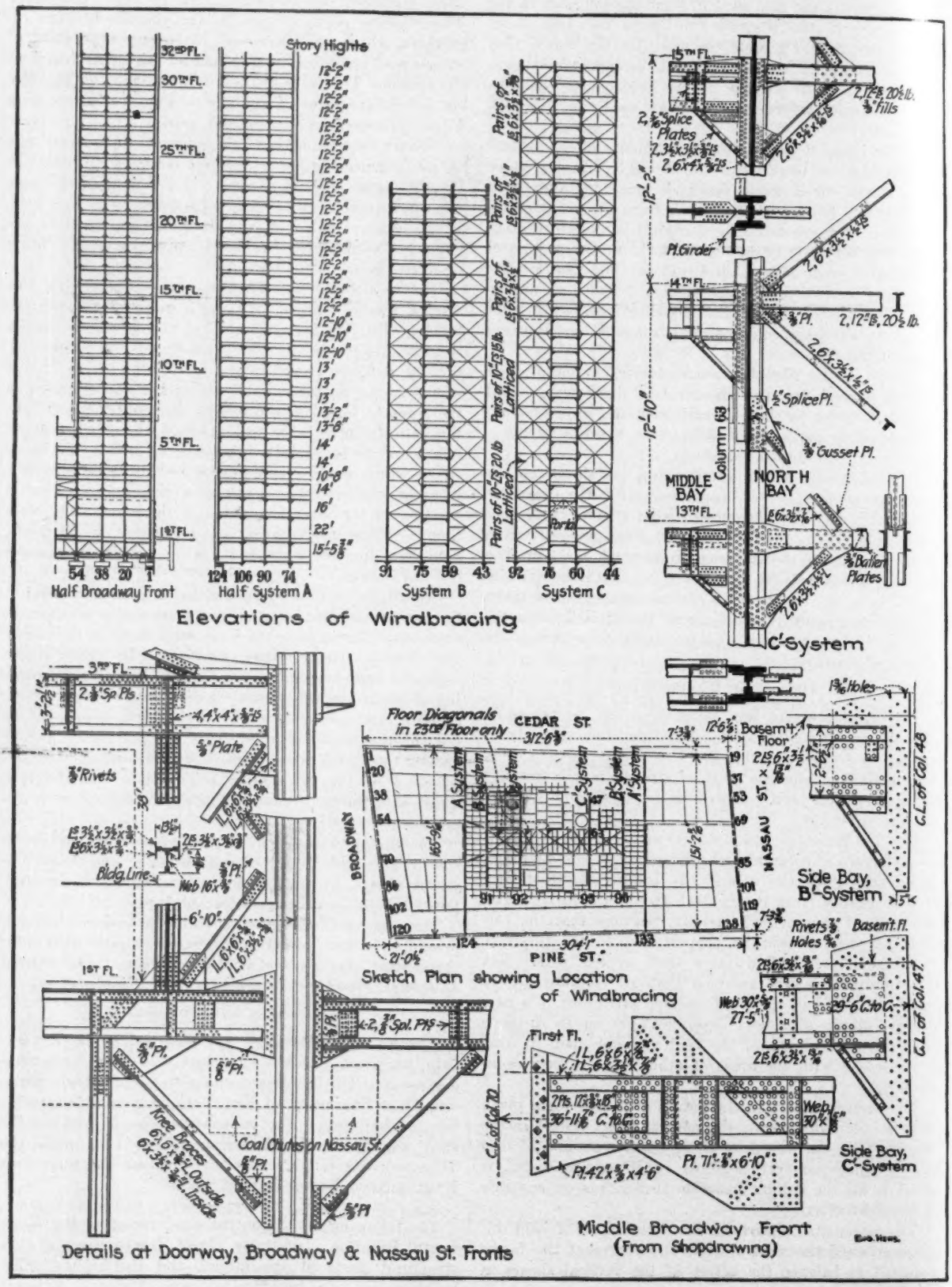
The distances between column stresses in successive stories gave the girder shears directly, and from the girder shears, by taking moments around the complete joint, the column shears were obtained.

The computation, according to the procedure outlined, proved to be well suited to convenient tabular work. In this entire analysis *the knee-braced bays were treated as though equivalent to the bays with diagonals.*

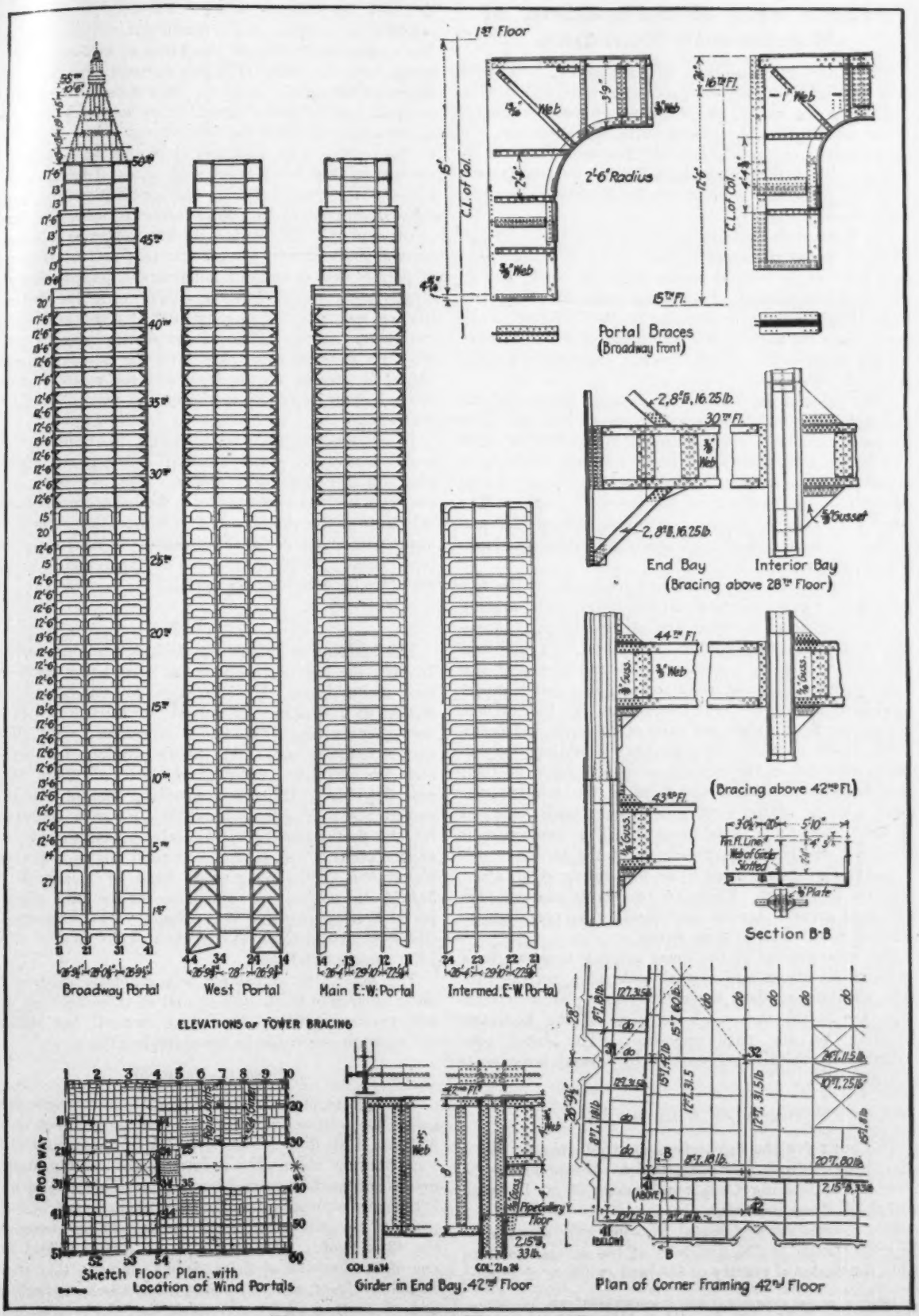
Designers and Builders

The Equitable Building was designed by E. R. Graham, architect, Chicago (Joachim G. Giaver, structural engineer). The Thompson-Starrett Co. (George Simpson, Chief Engineer), of New York, is general contractor for the building. The subcontract for furnishing the steel was let to the American Bridge Co., while the Thompson-Starrett Co. itself is erecting the steel (see Engineering News, Apr. 23, 1914).

The following article, on the windbracing of the Woolworth Building, facilitates direct comparison of the structural parts of the largest and the highest office buildings in the world. On pages 230 and 231 are given comparative windbracing drawings.



WINDBRACING OF THE EQUITABLE BUILDING



WINDBRACING OF THE WOOLWORTH BUILDING

(Plan and tower elevations are to same scale as on Equitable sheet).

Frame and Windbracing of Woolworth Building

The Woolworth Building, 233 Broadway, New York City, completed during the past year, is the tallest office building in the world, and the most prominent example of the tower type. Its general outlines are familiar, but the character of the windbracing has received little attention. We outline it in the following, for comparison with the preceding account of the Equitable Building windbracing.

The Woolworth Building is 55 stories high and extends 760½ ft. (base of flagpole) above curb. The main portion, however, is only 30 stories high, the upper half of the height consisting of a tower about 85 ft. square. From the windbracing standpoint the building is all tower, since the parts of the building outside of the tower are left entirely free of windbracing, the rearward wings excepted.

Great interest attaches to the bracing because of the thoroughgoing use of the portal system of bracing. This system has not been used extensively in windbraced buildings before this, whereas in the Woolworth Building it constitutes the principal or sole element of lateral strength in the building in the lower 27 stories. The upper portion of the building, involving lighter wind moments, is taken care of principally by girder-and-knee-brace stiffening.

The rear wings are braced independently of the forward half, which centers about the tower. The bracing of the wings is of the type first applied by S. C. Weiskopf in the Trinity Building, New York City, and subsequently adopted in a number of other structures* and today perhaps the most popular type, i.e., double-girder rigid-joint bracing without kneebraces. In this system, the regular floor girders are pairs of channels or I-beams (here, a pair of 24-in. plate girders) straddling the columns, and either the girders or splice-plates at their ends are riveted to the side faces of the columns, the shear moment of the rivet group being made sufficient to resist the girder wind-moment. The connection is developed wholly within the depth of the girder.

The two wings are made to act together by struts spanning the light-court. There are two struts at every fifth floor (see ground plan on inset sheet). No portal-girder action is counted on in these struts.

The arrangement of the tower bracing is shown by a series of elevations and a small-scale plan, on the full-page windbracing sheet herewith. (Both plan and elevations are to the same scale as those of the Equitable Building, on the page opposite.) The detail construction is typically represented by several larger-scale sketches.

Design of Wind Bracing

The basis for the calculation of wind stresses was a wind load of 20 lb. per sq.ft., which corresponds to the New York Building Code requirement of 30 lb. wind with 50% stress increase.

The *column stresses* caused by wind were assumed to be proportional to the distances of the several columns from the center of gravity of the bent or plane of bracing to which the columns belong. The *column shears* were assumed to be equal, in any particular story of the bent. The *column contraflexure* was taken at midheight of the story. On the basis of these assumptions, the girder shears in the several bays of a story necessarily came out unequal, and the girder contraflexure was found to be not at the middle point of the bay (except in the central bay of the bent). The designers state that this method of calculation was used because it gives stresses slightly higher than those based on the assumption of unequal column shears (girder contraflexure at middle of bay).

In applying the method to the east-west planes of bracing in the tower, the easterly pair and westerly pair of portals were considered independent, joined simply by struts in the middle bay; in other words, the east-west bracing was regarded as consisting of eight independent two-column bents. This, of course, applies only below the 28th floor, as above this level the two intermediate planes of bracing are omitted, and the entire east-west tower bracing is in the northerly and southerly walls of the tower.

In constructing the portal bracing, the upper or horizontal part of the portal was built as a separate erection piece, in the shape of a girder. The vertical portions also were built as separate pieces, field-connected by vertical joints to the columns and by horizontal joints to the upper part of the portal. The columns in most cases were constructed with projecting web or flange plates, as gusset-strips for attaching the portal.

Columns

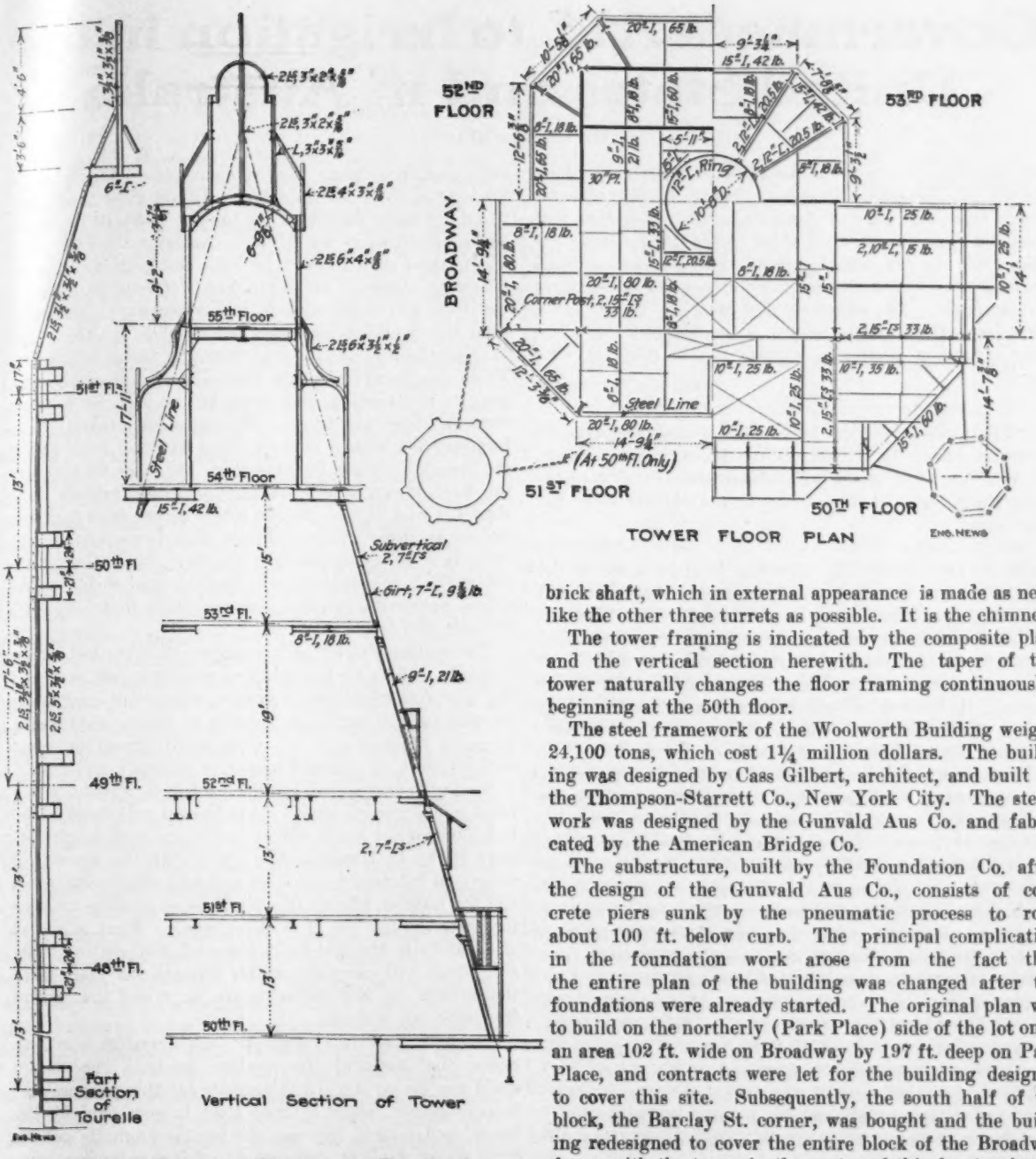
The building is notable for its heavy column sections. In fact, the heaviest columns far exceed any previously used in buildings. The maximum column has a cross-section of 700 sq.in. It is made of three webs, 21 in. deep, aggregating 12¼ in. in thickness; eight flange angles 6x6x⅞ in.; eight middle angles, 6x4x⅞ in., and two 43½-in. cover-plates each of total thickness 3½ in. In each cover-plate, one of the component plates is extended out 3½ in. as a connection for the portal bracing. Similarly, a longitudinal connection plate 7x1 in. is riveted at right angles to the middle of one of the flange-plate faces by means of two 5x4x½ -in. angles, as a connection for the other plane of portal bracing reaching this column. This column carries a maximum total load of about 4740 tons, of which 1300 tons is wind load.

The same type of column was generally used in the lower portion of the building; that is, three built-up webs and two cover-plates, forming a two-cell box column. All columns are made in two-story lengths.

Special Framing

The architectural development of the Woolworth tower called for setting back the wall line several feet at the 42d and 47th floor levels. At each of these points, the wall columns above do not have direct column support below, but must be carried by girders. This arrangement is typically represented on the windbracing sheet by a diagram of the girder-and-beam framing in one corner of the 42nd floor. Adjoining it is an elevation sketch of one of the heaviest of these girders, namely, that in the Broadway front, an 8-ft. plate-girder which supports the end of another girder on which the 42nd-story corner column is carried.

*For a recent example, see Consolidated Gas Co. Bldg., "Engineering News," Dec. 25, 1913, p. 1230, right-hand sketch of Fig. 16



TOWER FRAMING, WOOLWORTH BUILDING

Where the columns are thus offset, the transfer of wind shear in the outer faces of the tower must be made by way of the floor. The middle bay of the floor on eacn side of the tower has horizontal diagonals; for the rest the strength of the floor is counted on.

Corner turrets occur on the terraces where the walls are offset; those in the 42nd story are low, while those rising from the 47th floor are about 75 ft. high. Both have steel frame, supported upon and anchored to the floor steelwork. A vertical section through the larger turret is shown herewith. The northeasterly turret at this level is not steel-frame filigree work, but is a hollow brick shaft, which in external appearance is made as near like the other three turrets as possible. It is the chimney.

The tower framing is indicated by the composite plan and the vertical section herewith. The taper of the tower naturally changes the floor framing continuously, beginning at the 50th floor.

The steel framework of the Woolworth Building weighs 24,100 tons, which cost 1¼ million dollars. The building was designed by Cass Gilbert, architect, and built by the Thompson-Starrett Co., New York City. The steelwork was designed by the Gunvald Aus Co. and fabricated by the American Bridge Co.

The substructure, built by the Foundation Co. after the design of the Gunvald Aus Co., consists of concrete piers sunk by the pneumatic process to rock about 100 ft. below curb. The principal complication in the foundation work arose from the fact that the entire plan of the building was changed after the foundations were already started. The original plan was to build on the northerly (Park Place) side of the lot only, an area 102 ft. wide on Broadway by 197 ft. deep on Park Place, and contracts were let for the building designed to cover this site. Subsequently, the south half of the block, the Barclay St. corner, was bought and the building redesigned to cover the entire block of the Broadway front, with the tower in the center of this front. As the heavy columns, namely, those of the tower, were thus changed in location, the caissons had to be rearranged, and in some cases, where the proper arrangement was no longer possible, a combination of two caissons had to be used in place of one large caisson, and the columns supported by heavy triple girders spanning the two caissons.

The Total Value of the Mica produced in the United States in 1913 was $436,060. The production came from 11 states—North Carolina, New Hampshire, Idaho, New Mexico, South Dakota, South Carolina, Alabama, Virginia, Pennsylvania, Colorado, and New York, named in the order of the value of their output. The value of the production of mica in 1913 exceeded by $104,164 that of 1912, and was the largest ever reported.

Government Aid to Irrigation in the United States and in Australia

By Elwood Mead*

SYNOPSIS—In continuation of the discussion of "What Is the Matter with Irrigation," begun in the editorial columns of our issue of June 12, 1913, the author first pictures conditions in the United States which make farm ownership by the actual cultivators of the soil difficult in both unirrigated and irrigated areas, and particularly in the latter. The argument for State aid to irrigation farmers in the United States is supported by the practice and results in Australia, notably in the State of Victoria.

"The Answer to What Is the Matter with Irrigation?" contributed by E. P. Osgood to Engineering News, of Feb. 19, 1914, deals with obstacles to rural development in America that are encountered outside of irrigation areas.

Every increase in the price of land limits the numbers of those able to buy it. Farming land in some of the Western States now sells for five times as much as it did 10 years ago. Farms can be purchased within 30 miles of London at lower prices per acre, with less interest and better terms of payment, than is needed to buy farm land in parts of the Middle West. The result is that farm buying, in these localities, is now largely restricted to men with accumulated capital, while nonresident ownership and tenant farming are rapidly extending.

These are not desirable conditions, but if they are to be improved, it would seem that this country should bring into operation some of the agencies that are proving so effective in Europe, for enabling poor men to own the land they cultivate. The use of the credit and direction of the government in the buying and subdivision of land, the loaning of money to buy and improve farms on long-time payments, at less than half the rate of interest paid by farmers in Western America, are aids to the extension of agriculture and improvement of rural life now employed by nearly every civilized country excepting the United States, and they are as much needed in this country as in Ireland, if the United States is to continue to be a land of opportunity for farmers without capital.

The causes that restrict farm buying in the fertile East are delaying development in the irrigated West. The opportunities to build small, simple ditches and fill them from the mountain snows, without storage are all absorbed. Large and costly canals, with reservoirs, to supplement the summer flow of streams must now be built if irrigation is to extend. These mean costlier water-rights and larger annual operating charges, which must be paid in money instead of labor, as was formerly possible.

So long as there was public land to be had for the filing fee, and the cost of water-rights did not exceed $15 an acre, it was not difficult for a settler to bring enough land under cultivation to provide a living for his family and earn money to meet the payments on water-rights by working at intervals for others. Under these conditions, it did not mean disaster if the improvements of the homestead was slow, as was almost invariably the case. But when larger canals had to be built and more difficult engineering obstacles overcome, and water-rights began to cost from $40 to $50 an acre, it was necessary, in order to meet the payments on these water-rights, for the settler to bring the whole of his land promptly under cultivation. To do this, he must expend about $30 an acre on clearing, grading and seeding, and about $40 an acre for a house, farm buildings and fences. He begins his attempt, therefore, saddled with a debt of about $100 an acre, and if the money for these improvements has to be borrowed in the West, it can only be obtained on comparatively short time and at a rate of interest much higher than that prevailing in many other countries with less available capital. It is no longer possible, therefore, for settlers, without capital, to support their families and meet the required payments, and it is not surprising that fully 80% of those who do attempt this, fail.

The obstacles to irrigation settlement, described in Mr. Osgood's article, are becoming more serious each year, and the prospects of failure to settlers, with small capital, are so great and the hardships involved so severe, that the continuance of settlement of unimproved irrigation areas, without state aid, cannot longer be honestly encouraged.

To leave unchanged conditions which restrict settlement under government projects to men with from $4000 to $6000 capital would defeat one of the most worthy objects of the Reclamation Act. Men with this amount of money do not need to go West and help subdue the desert and the number who do this would be so small as to delay for years the full use of existing canals. What is needed is a carefully thought-out scheme of National or State aid, which will carry the settler through the years when the outlays for improvements are large and the returns from cultivation small.

The best place to inaugurate such a system would be under the National Reclamation works. The nation could use its savings' bank deposits for this purpose and thereby secure larger interest than is now paid by the banks, and largely increase the income from its canals. Such action, while an innovation in this country, is a common feature of irrigation development elsewhere. In the Australian States, where there are still large areas of public land which are practically free, the settlers are given financial aid to carry them over the lean years when they are improving their farms. Owing to the similarity of conditions there and in Western America, it is believed that the methods those States have adopted can be successfully employed here.

Such aid is a feature of irrigation settlement in every Australian State, as it is recognized that the outlay needed to bring unimproved irrigable land under cultivation is larger than is needed under rainfall. The most extensive application of this idea is found in the State of Victoria, where large areas have been purchased, subdi-

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vided and settled by men with too little capital to attempt to secure a home unaided. Under its scheme, men have succeeded who had not a dollar to start with and settlers with less than $1000 are more numerous than those with more than this sum, although oversea settlers with less than $1500 are not encouraged.

Knowing that these settlers must be helped and could not afford to lose time nor to make mistakes, the state worked out a plan for their management, as complete in its details as that of a corporate, private enterprise. In order to lessen their living expenses while securing a farm, the settlers are lodged in a state-managed rooming house where meals cost only 12c. and room and bed 20c. a day. They are shown the land available, by a responsible state officer, with no expenses for conveyances outside reduced railway fares. When the farm is selected, the state, if requested, builds a house for the settler, he having the choice of 14 different designs. When the settler selects his type of house, he pays a deposit, varying with the cost, and has twenty years in which to complete the payments. Some of these houses are built before the settler arrives. Where they are erected after the settler arrives, the state provides portable houses and tents to shelter the family, for a nominal weekly rental, so that in either case he goes to work at once. The state also grades and seeds one-fourth of each settler's holding. If desired, the settler pays only one-fifth the cost of this grading at once and has ten years in which to pay the remainder. At first all the grading work was done by the state, but now private contractors are doing much of the grading, but the state still pays four-fifths of the cost wherever it approves the contracts and accepts the completed work.

As a result of this assistance, many of the settlers who have gone into dairying have been getting cream checks which more than meet all living expenses in less than thirty days after selecting their farms. Even with this aid few settlers are able to bring the whole of their land into cultivation the first year. It usually requires two or three years to get the entire farm fenced, ditched, graded and seeded. During this development period the settler has a large outlay and less than full income and he would not be able to improve his land promptly unless further aid was extended. This is done by the state loaning the settler money on his improvements. Ditches, fences, grading of land, erection of buildings and the planting of orchards and vineyards are improvements entitling the settler to loans. The state values these improvements and lends up to 60% of the valuation. Sometimes this loan is given in cash and sometimes as credit against payments for land and water. At first, the state gave no aid in equipping farms. Now, it provides some costly implements which are loaned to settlers for a small charge. It keeps pure bred bulls in the dairy districts for the use of settlers and where the settler is known to be competent and reliable, it furnishes both horses and cows, the settler paying a deposit or giving additional security.

The extension of this aid would be hazardous if it was not accompanied by the oversight of settlers by capable, practical men. A scheme having so many generous features is certain to attract a considerable number who are indolent or impractical, and who seem to believe that the world owes them a living—or, in the language of Victoria, that it is the business of the state to spoon-feed them. Such settlers have to be promptly eliminated, and the Victorian Act provides for this.

There are other settlers whose lack of experience would result in costly mistakes if left to work out their own salvation. To guide and encourage the industrious beginner and weed out the failures, the state locates in each settlement an officer known as an inspector, who makes frequent visits to each farm, observes the progress made in improvements and cultivation and reports thereon to the commission in charge of the work. It is these reports which determine quite largely the extent of the financial aid which will be given a settler. If two-thirds of the farm is not brought under cultivation by the end of the second year further financial aid will be refused unless exceptional conditions cause the delay. At first, settlers are inclined to object to this oversight, as they regard it as interference with their private affairs, but it does not take them long to realize the value of being able to obtain competent advice free.

A good inspector soon brings his settlers to act together for their own good and for the reputation of the district. The emulation between settlements works wonders in hastening the cultivation of all the land.

The state retains the ownership and management of canals. During the first year, settlers are only required to pay for the water they use. After that they pay for the water allotted to their land, which rarely exceeds the water used. The price of the water is fixed so as to provide 4% interest on the cost of the works besides expenses of operation and maintenance. It is believed that this plan could be adopted with advantage in America on the Reclamation Service projects. If the ownership and management were retained by the government for 25 years and settlers required to pay only operating expenses and 4% interest on the cost, it would greatly reduce the earlier payments, and relieve settlers from the responsibility of management until they had gained experience.

In Victoria, settlers are given 31½ years in which to pay for their land, with interest at 4½%. The farms vary in size from two-acre blocks intended for farm laborers, up to holdings having a value of $12,500. The average-sized fruit farm at present is about 30 acres, and the dairy farm from 40 to 50 acres, experience having shown that these areas are all that the farmer and his family can properly cultivate unless considerable hired labor is used.

In equipping and improving these farms the state invests much more money than the settler, and it has to do this if the chance for a living is to be open to men of less than $1000 capital. The intention is to provide homes for farm laborers without capital and to enable farmers with a capital of $1000 to secure farms of 40 acres or less.

The following table will show the amount of money invested by the settler and by the state in a dairy farm of 40 acres, costing $60 an acre.

		Paid by settler	Paid by state	Period of deferred payments	Interest rate
Land	$2400	$144	$2256	31½ yr.	4½%
House	1200	150	1050	20 yr.	5%
Grading and seeding 15 acres	325	65	260	10 yr.	5%
2 horses	250	50	200	5 yr.	5%
20 cows	800	160	640	3 yr.	5%
	$4975	$569	$4406		

This does not take into account the expenditure for milking shed, outbuildings and fencing, but 60% of the latter outlay can be returned to the settler as a loan as soon as the improvements are completed, and this will pay for the material needed. One great merit of this plan

is the way it protects the settler from using up his capital for living expenses. One with $2000 who has to build his own house and grade and seed his land consumes about half his capital in living before he gets any return from cultivation, but with the house built and the alfalfa or other fodder crop sown before hand, the capital all goes into development.

In addition to the outlays mentioned, the settler must buy furniture for his house and harness for his team. He must have a wagon or cart, a cream separator and cans, a plow, a harrow, a mower and rake. He cannot buy these things and pay his deposits unless he has a cash capital of $1000 but the number who have $1000 is many times the number who have $4000 and under this scheme the capital needed is reduced with the acres taken.

Providing ready-made farms for settlers has brought about a larger extension in the irrigated area in the last four years than was obtained in the 25 previous years in which settlers were left to work out their own salvation, unaided. It has so increased the revenue from canals and the payments by settlers have been so satisfactory that the state is now considering a large extension of the irrigated areas and opening up for settlement areas of less than 30 acres, on still more liberal terms of payment.

The opportunities for extending such aid in America are greater than in Australia. Money can be obtained here at a lower rate of interest. In America there is a greater number of home seekers and settlers and they do not have to be brought half way around the world, as is the case in Australia. We also have more skilled irrigators, whose services could be obtained for purposes of instruction. The need is so great and the chances of success so encouraging that it is hoped state aid in creating ready-made farms will be tried on at least on project.

The Early History of Bituminous Street Pavements

Although the best known textbooks on pavements invariably state that asphalt or bitumen has been known and used for many ages—that bituminous cement was used for the walls of ancient Babylon—these same books are invariably hazy on the first use of asphalt during modern times for street paving.

The brief article in our issue of Apr. 30, 1914, p. 945, describing an experimental pavement on Oxford St., London, England, in 1839, gives a bit of the earliest history of bituminous pavements in England. Since that article was published we have found among the "Papers on Practical Engineering published by the Engineer Department for Use of Officers of the Corps of Engineers" an excellent monograph on "Bitumen," written by Lieut. H. Wager Halleck in 1840; and in view of the recent history of bituminous pavement and the patent fights over methods of construction we deem the following facts compiled by Lieut. Halleck worthy of the attention of present-day pavement experts.

Regarding the early use of native bitumen he writes:

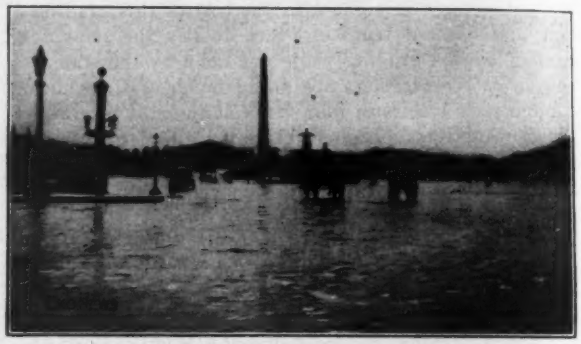
> For 30 to 50 years (written in 1840), mineral tar, in combination with sand and calcareous matter, has been used in Bordeaux and its vicinity for roads, pavements, floors and the roofing of houses. It was first used, with sand only, in the repair of roads near Bayonne, and being found to answer this purpose, it came afterward to be employed by builders, in its combined form, in preference even to slate and stone; and many houses at Bordeaux are now roofed with it.

Regarding the use of bitumen on macadam roads, Lieut. Halleck described the now so called penetration method as follows:

> The soil is first removed to a depth of about 5 in., according to the thickness to be given to the road covering, and the earth well rammed; to make it still more firm, small broken stones, about 1 cu.in. in size, may be driven in. A layer, about 4 in. thick of broken stone from ¾ to 1 in. in diameter is spread over and smoothed off so as to give the surface a slight curvature. Over this is poured a very hot and liquid mixture, of one-third coal tar, one-third pulverized asphaltic stone (of an inferior quality) and one-third fine sand. A sufficient quantity should be put on to penetrate all the crevasses of the stone and form a solid mass. After this has become cold, a layer of good bituminous mastic, about 1 in. thick, is applied, and, while still hot, is covered with a layer of broken stones about 1½ in. in diameter, placed close together, but without touching, and pressed into the mastic by an iron roller. The thickness of this covering may be varied, according to the wear and tear it will have to resist.

The first bituminous pavement laid down in Paris, it seems, was a genuine asphaltic block pavement, in many respects very similar to those now in use. On this subject, Lieut. Halleck quotes M. Partiot, Chief Engineer of Roads and Bridges of France in the "Annales des Pontes et Chaussées," for March and April, 1838:

> The first bituminous pavement in Paris was at the entrance of the Place de La Concorde. The pieces of stone [that is, artificial stone or compressed asphalt blocks] are



PLACE DE LA CONCORDE; SITE OF THE FIRST ASPHALT PAVEMENT IN PARIS

(From a photograph taken in the summer of 1913 by H. W. Durham, Chief Engineer of the Bureau of Highways, Borough of Manhattan, New York City. Now paved with wood block on a concrete foundation.)

> there arranged like mosaic work, with their smoothest side uppermost, and then connected together by a very adhesive bituminous mastic. They form a continuous and solid mass.
>
> This experiment has not been completely successful; for the road, which has been in use since Dec. 3, 1837, is already (March, 1838) worn away in many places; but this is the result of defects in the execution of the work. The contractor erroneously considered that his bituminous mastic was capable of resisting the direct action of the wheels and horses' feet. This mastic, which attaches itself very strongly to the stones, should be used only for cementing them together, so as to form one solid block, but it should not be visible, the stone alone being seen upon the surface. The stones of this pavement were not placed near enough together, and the intervals were filled with bituminous mastic, which is not capable of resisting friction; so that the joints are open, and the surface of the road, although at first smooth, has already become rutty and uneven.
>
> These defects might have been avoided if the following precautions had been taken. The fragments of quartz should have been placed as close together as possible, and the intervals between filled with small chippings of stone, which the larger masses would have sufficiently protected; these artificial paving stones should have been placed diagonally, in order that the direction of the joints should not have been lengthwise of the road, moreover the width of the joints

should have been only 0.3 to 0.4 in. instead of 1½ to 1¾ in., as they usually are. . . .

So far as ease of traveling or cheapness of draft are concerned, the bituminous pavement will always, without doubt, be preferable to the ordinary paved or broken stone roads. [These pavements cost about $2.53 per sq.yd.]

Lieut. Halleck describes further on in his monograph the construction and use of asphalt blocks, as follows:

Pavements have been made in Paris of fragments of quartz cemented together with bituminous mastic, and shaped by running them into molds. These blocks, properly prepared beforehand, are laid upon a bed of sand, and the joints, which are about 0.4 in. wide, filled with mastic [patented binder].

It appears from the foregoing that asphaltic pavements were laid in Paris in 1837; and from that time forward there was a gradual development, the heavy rolled compressed sheet asphalt, like pavement in use there today, which is, in most instances, a natural rock asphalt, was developed about 20 years later. But is also appears that asphalt had been used on roads in Bordeaux at least a quarter of a century before being tried in Paris. This is probably the earliest use during modern times of asphalt for road purposes. It is known to have been used in ancient times for mosaic pavements in buildings, halls and court yards, but so far as known it was never used on ancient roads subjected to wheeled-vehicle traffic.

Col. J. W. Howard, Consulting Engineer, New York City, in a lecture before the graduate students in highway engineering at Columbia University, on Jan. 24, 1914, gives the following information on the early use of asphalt:

In 1712, D'Eyrinys discovered, and in 1721 published, a description of the rock asphalt at Val-de-Travers, Neufchatel County, Switzerland, and several uses for this material. D'Eyrinys, in 1735, opened the bituminous limestone deposits at Lobsann (Germany), which deposit has been worked ever since. About 1797, the deposit at Seyssel, France, was opened, and then, like the others, was used for making asphalt mastic by extracting the bitumen from some of the rock and enriching the powdered fresh rock with it.

In 1802, Count de Sassenay took charge and reformed the methods, very much improving them, getting his supplies from the Swiss and French deposits mentioned. Rock asphalt-mastic was soon used for floors, sidewalks, public squares, bridges, and then streets. It was in Lyons, France, about 1834, that in mastic form, it succeeded fairly well for street paving. In the same year it entered Paris and rapidly became almost universally used for sidewalks.

Meriam began about that time to observe that lumps of the broken and powdered bituminous limestone, dropping from wagons near the quarries, were crushed and coalesced under wheel traffic into a firm, continuous sheet on the roads. It was then that the noted engineer, Leon Malo, of France, entered the industry and devoted his life to it. He is known throughout the world as the father of the asphalt industry.

Delano, of England, colabored with Malo in France, and introduced sheet-asphalt pavements, made of European rock asphalt, and asphaltic-mastic sidewalks, floors, etc., into England and the cities of almost all nations.

In 1854 Malo, after some experiments, laid the first successful sheet-asphalt pavement. It was made with compressed natural rock asphalt. He demonstrated the necessity of a firm concrete foundation. The wearing-surface layer was made by crushing, powdering, and then heating the bituminous limestone and spreading and compressing it in place on the street. This first street was Rue Bergere, Paris, which, from the first, was successful, and by proper maintenance it has been kept thus paved in constant good order for 60 years to date.

The success of compressed, rock-asphalt pavement in Paris since 1854, has attracted the attention of all important cities. London, England, began its use by paving Threadneedle St., in 1869, and has steadily extended its use to many other streets of that city. Geneva and Berlin began in 1871, and all the principal cities of Europe soon followed.

Its extensive use in America was prevented, first, because of the heavy freight charges from Europe, and secondly, because of the invention by DeSmedt of the American form of sheet-asphalt pavement. He laid his first piece on William St., Newark, N. J., in 1871, then in other cities, and in 1876 on a portion of Pennsylvania Ave., Washington, D. C., where his artificial composition of sand, limestone powder, and asphalt cement proved successful.*

Suggestions as to the Maintenance of Driven Wells, at Lowell, Mass.

Although in some parts of the country, driven wells are commonly used for public water-supplies, they are usually confined to relatively small cities. One of the larger cities with a driven-well supply is Lowell, Mass. In reporting on the possible treatment of the Lowell water for the removal of iron and manganese, F. A. Barbour, Consulting Engineer, Boston, Mass., made some suggestions regarding the maintenance and operation of the Lowell wells that may be of service to those in charge of driven wells elsewhere.

After speaking of the desirability of distributing the draft of water uniformly over the whole field, Mr. Barbour says:

To do this, it will be necessary to keep the well strainers in better condition than in the past—first, by washing down all wells, instead of driving, and second, by using an all brass well strainer section, instead of the iron and brass section now employed, in which the electrolytic action between the two metals contributes to the clogging of the strainer.

The present Boulevard system comprises some 450 wells, connected to a number of suction lines, which converge with the main pipe leading to the air chamber at the station. The several lines of wells vary considerably in age, and in the condition of the strainers, and thus it is that, under the vacuum maintained at the station, the new wells, or those last cleaned, have furnished the greater part of the water, and the quantity drawn from these new wells has been so large that the clogging of the strainers by silt is accelerated, and the quality of the supply rapidly depreciated by overworking the adjacent ground. The obvious remedy is to maintain the well strainers in reasonably uniform condition, and by partial closing of the gates to different degrees on the several suction lines, to so balance the water drawn from different parts of the well field that the water table will be more uniformly depressed—a larger area of subsurface sand being thus made effective in the filtration of river water, and the average rate of filtration by this increase in effective area being inversely reduced.

As already stated, in the past the greater part of the water has been drawn at high rates from a rather limited area, which shifts in location as the wells are cleaned, or new wells are driven. It is now proposed that when new wells are driven, the draft from these wells shall be limited to the quantity which they can reasonably continue to supply without too rapid clogging of the strainers or overworking of the adjacent soils; that the quantity drawn from each section of the field shall be determined by pitometer gagings, and that these gagings, together with observations of the elevation of the water table in test wells, shall be made the basis of a system of control such that the total area of the well field will be more uniformly worked.

It is necessary to recognize that the continued maintenance of a driven-well system requires thorough development of the wells when driven, the use of material in the well points which will not corrode, and a control of the draft from the different sections of the field which will, as far as possible, not overwork the soil in any one section to the detriment of the quality of the water. By the methods proposed, and which can be carried out under the supervision of the operator in charge of the purification plant, it is believed that a greater quantity of better water than has been obtained in the past can be drawn from the territory in which wells are already driven, and that the continuous increase in the amount of iron and manganese, which has been in evidence in the past, will be checked.

*Authors or authorities on bituminous limestone and its uses. Busch, about 1802, d'Aoust (1834), Purivs (1836), Itier (1839), Millet (1840), Huguenet (1847), Daubree (1850), Hessell & Kopp (1855), Benoit (1860), Homberg (1865), Leon Malo (1866), and his subsequent publications, Delano (1871 to 1893), Jeep (1867), Meyn (1872), Colgnet (1875), Kayser (1879), Woas (1880), Zutter (1880) Ducker (1881), Wein (1881), Dietrich (1882), Schubarth (1883), Narcy (1889), Baumeister (1890), Lonholdt (1891), Jaccard (1895), Haft (1897), Loewe (1906), Boorman (1908), Danby (1913).

Reconstruction of a Jersey City Railway Passenger Terminal

SYNOPSIS—Reconstruction of a railway and ferry terminal handling about 50,000 passengers per day. Existing buildings and ferry slips were either replaced or remodeled and the whole terminal enlarged. High trainshed was replaced by a large shed of the low type. The site is filled ground of depth up to 110 ft.

The complete reconstruction of the Jersey City passenger terminal of the Central Railroad of New Jersey, in progress since 1911, is being rapidly brought to conclusion. In addition to the passenger service of the New Jersey Central, the New York passenger traffic of the Philadelphia & Reading Ry., the Baltimore & Ohio R.R., and the Lehigh Valley R.R. is handled through this terminal. The Lehigh Valley was taken on in 1913, during the heaviest construction. About 50,000 passengers are handled per day and most of the business is short-haul commutation business.

style steel trainshed and erection of a low shed covering a much larger area. Fig. 1 shows the extent and scheme of this work. The table gives dates and rate of progress on the several elements.

A general idea of the size of the work may be had from the following figures: the ground area occupied by the station and ferryhouse is 2.31 acres; by the train concourse, 0.52 acre; by the new trainshed, 7.18 acres.

Double-Deck Ferryhouse and Slips

An old one-story wood structure was torn down in May, 1912; and in June, 1914, a new and modern steel-frame two-stay building with reinforced-concrete floors, terra-cotta block partitions, a concrete roof covered with tar and slag, and an ornamental copper front, was erected. In addition to the ferryhouse, four slips with pile racks were constructed. With this work commenced the active construction on the passenger-handling section of the enter-

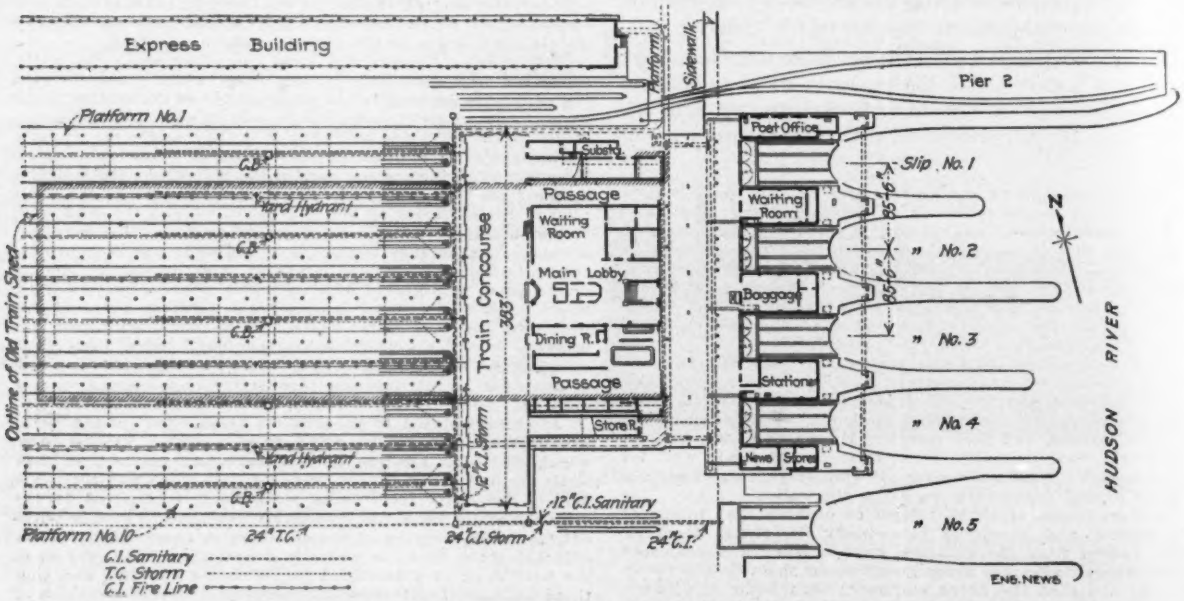


Fig. 1. General Layout of Ferry House, New Station and Trainshed

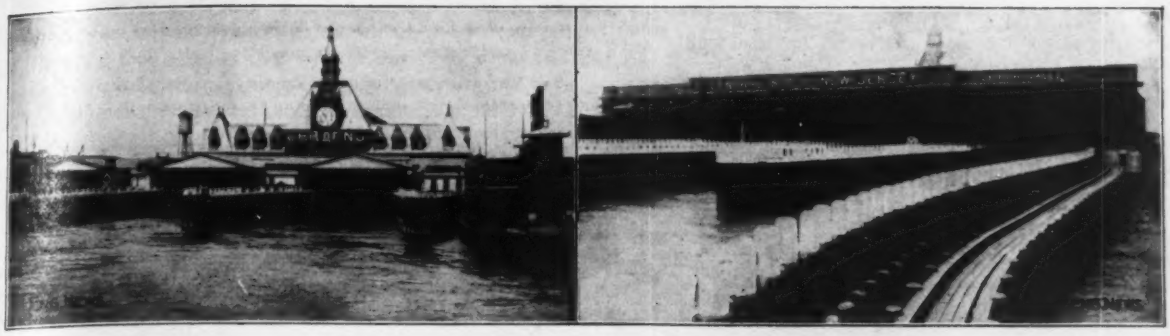
While the work was in progress about 185 trains per day were handled in each direction on the two main-line tracks and the two tracks of the Newark branch—four passenger tracks in all. The passenger-coach yards, the locomotive terminal, the U. S. Express Co. offices, the Pullman yards and service buildings were on the north side of the main line.

The work comprised the following elements, proceeding west from the river bank: Erection of a new two-story double-deck ferry house with four slips; rearrangement of the old station buildings; building a three-story steel, brick and concrete building, 800x56.5 ft., for handling express business, offices, records, etc.; the erection of a service building for power purposes, e.g., heating, cleaning service, water and compressed air; rebuilding two small piers in the North River; demolition of the old-

prise, through all of which traffic was not interrupted. To maintain normal service two temporary slips, one on each side of the existing four, were built and the two northern slips abandoned, leaving four still in operation. The buildings between these two northern slips were then torn down, part of the ferry concourse was closed, and the foundation work on the northern half commenced.

When the northern new slip had been rebuilt and placed in service, old No. 3 slip was taken out of service, and when the ferry bridge of new No. 2 was put in service old No. 4 slip was demolished, as was also the southern half of the old ferryhouse. The foundations for the southern half were placed in a manner similar to that employed on the northern half. All the foundation work was completed in December, 1913.

The ferry bridges are of the wooden bowstring truss



FIGS. 2-3. OLD AND NEW FERRY HOUSES AND SLIPS

OLD CONSTRUCTION NEW CONSTRUCTION

type, 79 ft. long, with one end supported on a rocker beam. The off-shore end is raised and lowered by electric-driven machinery, supported on steelwork entirely separate from the building. The raising and lowering of the bridges to fit varying conditions of tide and boat-loading is accomplished at a speed of about 6 ft. per minute.

The ferryhouse is fireproof throughout and rests on about 5000 piles 60 to 80 ft. in length with a butt diameter of about 14 in., driven through soft mud to rock. The building contains 1500 tons of structural steel, 7000 cu.yd. of concrete, and 1,800,000 ft. b.m. of lumber in the foundation under water and in the ferry racks.

The contractor's plant for the foundation work consisted of two floating piledrivers and two floating derricks. The land piles (west of the bulkhead) were driven by a standard land piledriver equipped with a 1½-ton drop hammer.

Gravel, sand, etc., for the substructure were delivered on scows from which they were lifted by a floating derrick to a large concrete mixer. This mixer was situated on a fixed platform supported by piling, about 20 ft. east of the bulkhead. From the mixer it was lifted in a steel tower about 60 ft. high to chutes suspended by wire cables. These chutes were shifted as desired, during the work. Materials for the superstructure were delivered by cars to a mixer located on land, from which the concrete was conveyed by wheelbarrows to a hoist and lifted to the upper floor and roof.

During the latter part of 1912 and the first part of 1913 when the foundations were under way, the work was pushed hard; as many as 400 men per day were at work, representing weekly payroll of approximately $9000. This work was prosecuted in two shifts of 8 hr. each.

The steelwork was furnished by the American Bridge Co. G. B. Spearin, of New York City, placed the substructure and erected the steel for the new ferryhouse. The superstructure was erected by C. T. Wills, Inc., of New York.

REARRANGING OLD STATION BUILDING

The old station was a three-story brick building on pile foundations. It was constructed about 1888, and not

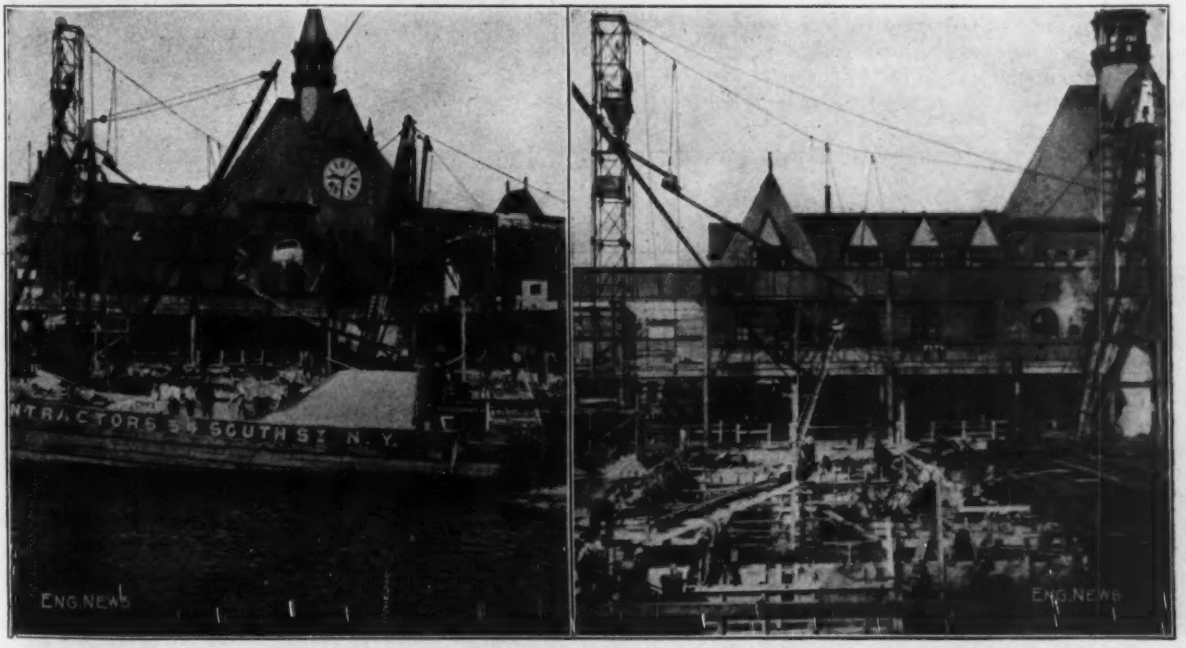


FIG. 4. CONTRACTOR'S PLANT ON FERRY HOUSE CONSTRUCTION.

well suited for present needs. It was found, however, that the same outside walls could be used; only a rearrangement of the first floor interior was necessary. Part of this floor was formerly used for conductors' and brakemens' layover rooms, stores and records, ticket offices, baggage, mail rooms and other various uses. This floor was completely remodeled and given over entirely to passenger business. The upper two floors were left practically unchanged.

FIG. 5. A COMPLETED SLIP AND A PORTION OF NEW FERRY HOUSE

A small interior structure, 46x20x10 ft. high, was constructed in the center of the old waiting room to be used for ticket offices, information bureau, telegraph office and parcel room. The first floor of the station building is divided into three general bays; the central portion (formerly the waiting room) is now called the main lobby. The entire south wing is given over to restaurant, café and grill room. A tiled floor, tile wainscoting and plaster side walls and ceiling replace the old wood floor, wainscoting and ceiling. The rearranged north wing is given over entirely to waiting rooms, smoking room and toilets. The main waiting-room has a terrazzo floor laid on concrete arches, with green-tile wainscoting and plaster walls and ceiling. It is lighted by a large leaded-glass skylight in the roof as well as by large windows opening on the train concourse and main lobby.

A feature of the station improvement is the wide central stairway in front of the ticket offices leading from the main lobby to the upper deck of the ferry concourse. The main lobby was given a new red-tile floor laid on the steel and concrete base of the old floor; and the whole inside of the station was cleaned, repainted and refitted.

Another important feature is the addition of two inclines, one on each side of the station building, leading from the train concourse to the upper-deck ferry concourse. They slope 1 in 10, and have cement floors with carborundum finish to prevent slipping. They were thrown open to passenger traffic April last.

North of the old depot was constructed a new two-story building containing the station master's office, the trainmen's layover room and a substation for light and power.

SERVICE BUILDING

The service building contains the heating plant, consisting of six 250-hp. boilers for station and yard heating; a fire pump of 90,000 gal.-per-hr. capacity; vacuum-cleaning machines for cleaning the station and the cars in the yard; and a 2500-cu.ft.-per-minute air compressor for train charging for airbrakes and for train-signal operation in the terminal.

CONSTRUCTION OF TWO PIERS

The work of rebuilding two railway piers just north of the ferry slip has not yet been commenced. Pier No. 2 is intended for the railway-mail service, which has greatly increased during the past two years on account of the addition of the parcel post. Here also will be delivered westbound mail from incoming steamships, thus saving several hours on this class of matter. Pier No. 1 is used for the emigrant service, and will continue to handle this travel after remodeling.

LOW TRAINSHED

The outline of the old trainshed is superimposed on Fig. 1, which shows the new layout. The old structure was 512x216 ft. in plan, with a steel-truss roof 142 ft. 7 in. in span, truss-spacing 32 ft., 70 ft. high from top of rail to peak. There was also a small lean-to 36 ft. 6 in. wide on each side. Within this shed were 12 tracks of 90 cars capacity. The old shed was built in 1888 and re-

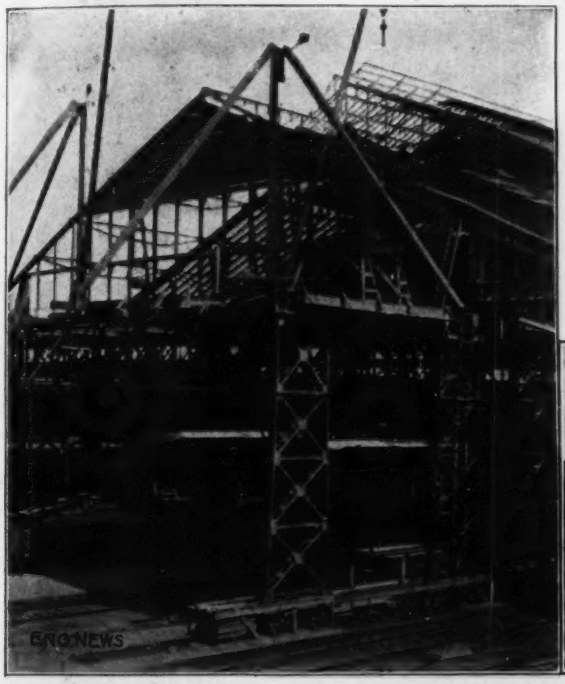
FIG. 6. NEW TRAINSHED AND STATION, FROM THE WEST

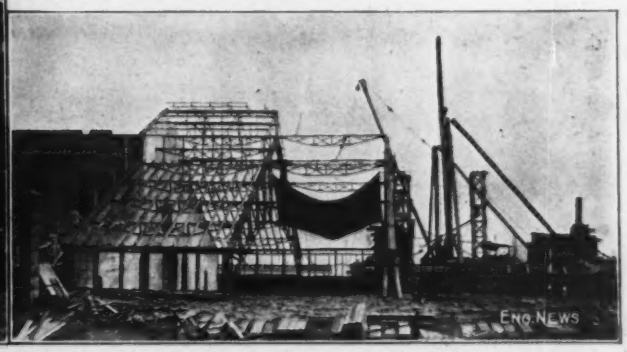
(The long building on the left is the express building. The sky-line of New York City is in the distance.)

quired for its maintenance a permanent gang of painters on account of the corrosion due to locomotive gases.

In July, 1912, the work of replacing this old shed with a shed of the Bush type was started. The new structure covers an area of 817x383 ft. and has 20 platform tracks with a capacity for 225 cars. The roof is supported on round cast-iron columns spaced 27 ft. c. to c. along the platforms. There is an 11-ft. cantilevered roof on each side. The distances from top of rail to top of platform and from top of rail to underside of smoke duct are 6 in. and 16 ft. 3 in., respectively.

Drainage is furnished by wrought-iron pipe leaders carried inside the supporting columns. These leaders run to a terra-cotta drainage system, situated between alternate pairs of tracks. This system discharges into a 3-ft. trunk sewer, south of the station and is carried thence to the North River.





FIGS. 7 AND 8. TRAVELER STARTING REMOVAL OF OLD TRAINSHED

(The traveler was built of three standard 8-track steel signal bridges, mounted on a timber frame on wheels, and covered with a continuous platform. The first bridge carried the blocking; on the second were two derricks; the third carried the boilers and hoisting engines for operating the derricks.)

Between tracks in the shed are steel intertrack fences, and the platforms are blocked off at the train concourse by a sliding-panel fence. At the rear of each track and facing the station are Hutchinson train indicators, which display on large banners the departing time and station stops of the trains.

Between the shed and the station is a concourse, 58 ft. 6 in. wide, running the whole width of the train shed. This is a striking feature of the whole construction. It is separate from the station building and is constructed of heavy steel-arched girders resting on steel columns, supported by piles. Along the center of the roof runs a self-ventilating skylight 19 ft. 6 in. wide. The face of the structure above the trainshed roof is fitted with side lights in movable steel sash. The ends of the concourse are inclosed in glass; the lighting and ventilating arrangements are very liberal.

The reinforced-concrete platforms and concourse are carried on reinforced-concrete girders, supported on concrete piers which rest on wood piles below groundwater level. About 4000 piles were used in the foundations of the shed and concourse. These piles are from 50 to 110 ft. long, the longer ones being spliced. The splice is made by means of 24-in. steel dowel-pins 1 in. in diameter or by an iron collar 12 in. in diameter and 24 in. long, of ½-in. metal, for reinforcing the joints between pile and follower. The tracks themselves are carried on the natural surface, which is all filled ground, the fill having been made some years ago when the old trainshed was built.

During the progress of the work the pier excavations were kept dry by means of hand pumps and portable gasoline-driven pumps. Most of the excavation was done by hand, but later in the job a locomotive crane with an orange-peel bucket was used.

Piledriving was commenced on the south side of the old trainshed in July, 1912; and shortly afterward on the north side. Early in 1913 platforms 10, 9, 8, 1 and part of 2 with shed roofs 1, 8 and 9 were completed. The new tracks covered by these roofs were completed and connected with the interlocking system in March, 1913. These tracks were located just north and south of the old trainshed, traffic inside the shed therefore not being interrupted as yet.

On Apr. 8, 1913, the removal of the old shed was commenced, the large central portion being taken down first, then the southern leanto. Meanwhile work was started on platform 7.

The work was pushed from south to north. New platforms were put in service as old platforms were discontinued. The work was carried on in pairs, i.e., when a new shed covering two tracks had been finished, the tracks were put in service and two old ones were taken out. When the work was started, 14 tracks were in service. It is worthy of note that thereafter, 16 tracks were always in service and on Saturdays and holidays, from 17 to 19 tracks could be used.

Getting material to and from the site without delaying the 370 trains per day and the many drill movements between the locomotive terminal and the trainshed, was no small problem. Work was carried in two shifts of 8 hr. each; the number of men at work reaching 450, involving a payroll of about $10,000 per week.

The plant used in this work consisted of two standard piledrivers, one with a 1½-ton drop hammer, the other with a 2-ton steam hammer. The rate of driving averaged about 10 piles for each piledriver per 8-hr. shift; being largely affected by the large area and the small concentration of work in progress within this area, due to its dependence upon traffic conditions.

Concrete was mixed by two mixers located on flat-cars. For the piers, girders and platforms, the concrete was chuted to place. The concrete for the roof was delivered from the mixer through a short chute to the bucket of a wooden hoisting tower situated on an adjacent flat-car. From this tower, concrete was distributed by means of three chutes fastened together and supported by two derrick booms projecting from the tower. This rigging was very flexible and could be quickly moved from point to point.

The steelwork and cast-iron columns for the shed framing were delivered to the job on cars and lifted directly into place by a small locomotive crane, having a 36-ft. boom. The steel for the large concourse was placed by a larger crane having a 70-ft. boom. The latter work required great care on account of the surrounding passenger traffic. This crane handled steel members weighing as high as 13 tons, from very awkward positions.

The construction of the trainshed, train concourse and platforms involved the driving of 4000 piles, the placing of 15,000 cu.yd. of reinforced concrete; 30,000 cu.yd. of excavation; 6000 lin.ft. of terra-cotta drain, and about 2700 tons of structural steel.

The general contractor was R. P. & J. H. Staats, of New York.

Method of Removing Old Trainshed

The 14 steel-roof arches of the old trainshed carried a slate roof laid on wood purlins and roofboards, and a large central skylight and ventilator. In taking down the shed, the skylight, ventilator, slate and roofboards were removed first and lowered by hand to cars on either side. To take down the heavier members, a traveler was erected, consisting of three 8-track, steel signal bridges with steel bents supported on a timber frame mounted on six wheels, three per side, running on one rail on each side, laid on two of the old concrete platforms. Opposite wheels were 108 ft. apart, and the clearance between the shed roof and the top of the traveler was 24 ft.

The traveler was erected just west of the old station on a Sunday morning without interference to traffic. Two locomotive cranes placed the leg frames while the signal bridges were being lifted one at a time from flat-cars by a wrecking crane and swung at right angles across the tracks. The two locomotive cranes hooked on to a signal bridge, one at each end, and lifted it to position, where it was bolted to its legs. After the three bridges were placed they were joined together and a light, timber platform laid over them.

Fig. 9. General View of New Trainshed Construction, Looking West

(Semicompleted sheds with tracks in operation; roof 6 under construction, showing concreting tower on flat car, and the concreting mixer under roof 6 for concrete-platform slabs; pile drivers working on platform at right of picture; trenching for drainage mains between tracks; temporary wooden butterflies over tracks in operation, at extreme right.)

FIG. 10. INTERIOR OF NEW TRAINSHED

GENERAL ORDER AND PROGRESS OF WORK

Element	Start	Finish	Time in Months
Express building	May, 1911	Dec., 1912	19
Ferryhouse, 4 slips, etc.....	May, 1912	June, 1914	25
Rearrangement of passenger station	July, 1912	June, 1914	23
Service building	July, 1912	Dec., 1912	5
Train sheds	July, 1912	Feb., 1914*	19

*Six bays or twelve-track width of the shed not to be entirely completed until new interlocking tower is in service, probably early in 1915.

progressed, wood butterfly sheds were erected on three platforms to protect passengers from the weather.

Greater difficulty was experienced in dismantling the traveler than in its erection, on account of the wood butterfly sheds put up in the meantime. The same general method and plant, however, were employed. The bridges were unbolted, one at a time. Locomotive cranes then hooked on to each end and moved westward under their own power, carrying a bridge with them above the butterflies. At a point west of the butterflies the bridge was lowered to the ground. An erecting crane then swung it

FIG. 11. INTERIOR OF TRAIN CONCOURSE

around and onto a car. Meanwhile, the locomotive cranes had returned for another bridge.

The operations of assembling and dismantling the traveler required but about four hours each. This removal work was done by Terry & Tench, of New York.

The first section of the platform carried the blocking; on the second section were two derricks for lowering the steel; while on the third were the boilers and hoisting engines for operating the derricks.

In operation, the traveler moved eastward. The forward bridge was run under a truss to be removed; the wood roof beams, steel purlins and the steelwork of the monitor were lowered to cars underneath. The truss was then blocked up and cut into section by the oxyhydrogen flame. About 870 tons of old steel were removed. After removing the main shed, the steelwork of the north leanto and later the south leanto was removed by a locomotive crane.

The traveler was erected Sunday, Apr. 13, 1913, started removing material Apr. 20, and was dismantled on Sunday, May 21. This work was accomplished without delay or accident. Trains were run under the traveler which spanned six tracks, and on both sides of the traveler. Two tracks were used for loading the steel removed. Canvasses were hung to protect passengers from falling rivet heads, small pieces of steel and dirt; and as the work

CENTRAL RAILROAD COMPANY ORGANIZATION

Reconstruction work was accomplished under the supervision of the Engineering Department of the Central Railroad of New Jersey: J. O. Osgood, Chief Engineer; A. E. Owen, Principal Assistant Engineer; J. J. Yates, Bridge Engineer; A. M. Zabriskie and H. E. Van Ness, Assistant Engineers of Design; and C. M. Titworth, Assistant Engineer in Charge of Construction.

Government Forest Reservations in the Eastern Appalachians now amount to 1,104,000 acres, for which $5,556,000 have been paid. Of the land purchased up to date, about 133,000 acres is in the White Mountain area in New Hampshire and the balance is in various parts of of the Southern Appalachians, from Virginia to Georgia. About $2,000,000 of the original appropriation remains available for additional purchases during 1915.

The Effective Width of Reinforced-Concrete Slabs Supporting Concentrated Loads

By C. R. Young*

The frequency with which reinforced-concrete slabs are subjected to concentrated loading is sufficient to warrant careful determination of the lateral distribution of the loads. Practically all loads on highway-bridge floors are of this class, and although the floors of buildings are usually designed for uniform loads, the actual loads are applied over a relatively small area of the floor surface.

Nevertheless, uncertainty and considerable divergence of opinion still exists among engineers concerning the proper methods of proportioning slabs thus loaded. The method adopted is usually based on a more or less rational guess at the probable lateral distribution of the load and then an inch or so is added to the calculated thickness "to be on the safe side and to allow for wear." Building codes and general specifications for the structural work of buildings are practically silent on the matter and few highway-bridge specifications make any mention of it. The engineer is thus left largely to his own judgment in making loading assumptions that not only greatly affect the proportions of the slabs themselves but have an important effect on the weight and cost of the structure as a whole.

The only safe basis upon which to found any rules indicating the lateral distribution of concentrated loads must be experimental. For this reason, the results of such tests as appear to have any bearing on the matter will be briefly reviewed, and from them general distribution rules will be established, giving the effective width in accordance with the experimental evidence available at the present time.

In June, 1913, A. T. Goldbeck, Testing Engineer of the United States Office of Public Roads, presented a paper to the American Society for Testing Materials setting forth the results of a series of tests of reinforced-concrete slabs subjected to concentrated loading. These slabs varied from 4 to 7 in. in total thickness and, with the exception of one 6 ft. wide and 13 ft. long, were all 7 ft. wide by 9 ft. long. Loads were applied centrally over a bearing area 6 in. in diameter in case of the latter slabs and by means of one-half of a vitrified paving block for the 6x13-ft. slab. Careful observations made up to the point of failure indicated that at a distance of as much as 4½ ft. on each side of the central load, the slabs exerted considerable resisting moment, even in cases where no transverse reinforcement was provided. While hesitating to draw conclusions from what seemed to him insufficient data for generalization, Mr. Goldbeck stated that in slabs of the kind tested and from 3 to 6 ft. in span the concentrated central load might be safely considered as carried by a width of slab equal to not more than 0.8 of the span length and the moment might be regarded as uniformly distributed over this "effective width."

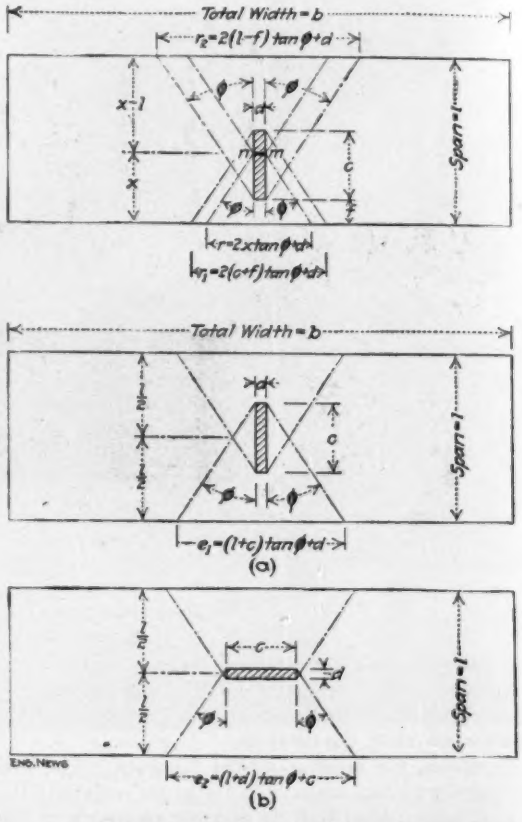
W. A. Slater, first assistant of the Engineering Experiment Station of the University of Illinois, in discussing Mr. Goldbeck's paper, cited a related series of tests being carried on at the University of Illinois. The tests then completed were made on (a), a large number of beams having a span of 4 ft. and widths varying from one-half the span up to twice the span; (b), one beam having a span of 10 ft. and a width half as great, and (c), two beams having a span of 30 in. and a width of 3.2 times as great. The loads were applied at the one-third points of the span.

On the basis of the latter tests and of those reported by Mr. Goldbeck, Mr. Slater recommended that where the total width is greater than twice the span, the effective width e be assumed as

$$e = {}^4/_3\, x + d$$

Where x = the distance from the edge of the concentrated load nearest the support considered to that support;

d = width, at right angles to the span over which the load is applied.



Figs. 1-3. Diagrams Showing Position of Concentrated Loads on Slabs

For slabs having widths less than twice the span, reductions in the effective width are recommended by Mr. Slater in accordance with a curve plotted from the results of the tests made at the University of Illinois. It was pointed out that, so far as tests had indicated, the effective width was but little influenced by the depth of the slab or by the percentage of longitudinal reinforcement. The introduction of transverse reinforcement would, no doubt, increase the effective width but this matter had not been investigated.

A striking conclusion arising from these results is that since the effective width for point loading is a constant

*Assistant Professor of Structural Engineering in the University of Toronto, Toronto, Ont.

proportion of the span, the depth of slab required to support a concentrated load would be approximately the same for all spans.

Confirmation of these conclusions with respect to the width of slab over which a concentrated load is distributed is afforded by analysis of another series of tests undertaken for quite a different purpose. In his "Discussion of the Basis of Design for Reinforced-Concrete Floor Slabs," in the *Proceedings* of the National Association of Cement users for 1911, A. R. Lord draws attention to a number of tests made at the University of Illinois in order to throw light on the lateral distribution of stress in wide beams. This is an important consideration in the design of girderless floors, since it is desirable to know whether only those rods of a belt which pass over the column capital may be taken into account in figuring the resistance to negative bending moments, or whether some or all of the remaining rods of the belt may be included. The beams tested were 3 ft. wide, 4 ft. 10 in. long and 3 in. deep to the reinforcement. They were loaded across their full width at the ends and were supported at the third points, in some cases for their full width, in others for one-half of their width, and in still others for one-fifth of their width. The beams supported for half their width carried, taking the average result, the same load as those supported for their full width. Those supported for one-fifth of their width, and without transverse reinforcement, carried 14.4% less load than those supported for their full width, but for two other beams supported in the same way and provided with 0.4% of transverse reinforcement, the falling off in the load carried was only 3%.

If, now, the wide beams thus tested be considered *upside down*, they would become simple beams of approximately 4 ft. 8 in. in span loaded at the third points with loads bearing, in some cases, on only one-fifth of the width of the beam, in other words, with concentrated loads. From the loads actually carried, it is evident that the moment arising from such concentrations was distributed fairly well over the width of 36 in., even in the case of those beams without transverse reinforcement, and with surprising uniformity for those transversely reinforced. It follows that what has been called above the "effective width of the slab" would, on the basis of the tests just cited, be about 31 in., or 0.55 of the span of the slab. According to the rules given by Mr. Slater, the effective width would be about 25.5 in. or 0.46 of the span of the slab.

Further light is thrown on the subject of the lateral stiffness of slabs by a study of the Tests of Reinforced Concrete Footings conducted by Prof. Arthur N. Talbot at the University of Illinois Engineering Experiment Station and reported in Bulletin No. 67 of that institution. For a series of column footings 5 ft. square supporting a central pier 12 in. square it was observed that "about three-fourths of the steel is effective in resisting the calculated bending moment, or rather the stress in the highest stressed bars is the same as if three-fourths of the steel bars, equally stressed, made up the resisting steel." According to this, the effective width of the footing slab would be 45 in. or 0.75 of a hypothetical span equal in length to the side of the footing. Applying Mr. Slater's rules to this case the effective width would be 40.3 in., or 0.67 of the span.

A series of tests which may be expected to solve many problems connected with this matter was commenced by Prof. C. T. Morris of the Ohio State University in 1911, but no report has been issued further than a statement by Prof. Morris that the results conformed in general with those reported by Mr. Goldbeck.

With the object of affording some guidance to designers in estimating the effective width of slabs supporting concentrated loads, the writer submits below an analysis of the problem and working formulas derived therefrom based upon the experimental results already described.

Let a concentrated load, bearing on an area of length c and width d be applied in any general position to a slab having a total width b not less than twice the span length l, as shown in Fig. 1. If any small element of this load, as mn, distant x from the nearer support, be considered, it is directly evident from the experimental investigations cited above, that such part of it as is delivered to the nearer support may be regarded as wholly transferred to that support within lines drawn through the end points of the element making angles ϕ with the direction of the span of the slab, the tangent of which angle is between 0.67 and 0.80. For brevity, the angle between these two bounding lines, that is 2ϕ, may be called the "effective angle." The reaction at the near support may thus be regarded as uniformly distributed over a width of $r = 2x \tan \phi + d$, or if $\tan \phi$ be assumed as 0.67, a conservative value in the light of experimental results, this width becomes $1.33\,x + d$.

Although there is no direct experimental evidence that the effective angle to the farther support is the same as to the nearer, careful consideration of the matter leads to the conclusion that it is. If the farther support were moved in so that the element of load, mn, became central, the effective angles to the two supports would be equal. There would then be applied to the shifted support along a length that may be called the effective reaction-width of the reduced span a uniformly distributed reaction which, when the first support was replaced in its original position, would become a concentrated load on the original span. The virtual load would be delivered to the support nearest to it, but farthest from the original eccentric element of load mn within the same effective angle as would apply to the nearer support in any case. It thus appears that the effective angles from any element of loading to the two supports are equal.

When the selected element of loading is not at the center of the span and the reactions at the two supports are thus not distributed over the same effective widths, the question arises as to what should be the effective width of the slab for the calculation of resisting moment. It is probable that the average of the two reaction-widths would be the correct width to assume for this purpose, and in the absence of direct experimental evidence on the matter, this assumption is recommended.

Considering as a whole the concentrated load shown in Fig. 1, it follows that the part of it delivered to either support may be regarded as wholly transferred to that support within lines drawn through the remotest corners of the loaded area and making angles with the direction of the span of the slab equal to ϕ, one-half of the effective angle. If f be the distance of that edge of the loaded area nearest a support, from the support, the effective reaction-width, r_1 for the nearest support would be

$$r_1 = 2\,(c + f)\,\tan \phi + d$$

and for the farther support it would be,

$$r_2 = 2\ (l - f)\ tan\ \phi + d$$

As in the case of an element of loading, the effective width of the slab for the computation of moment of resistance may be taken as the average of the reaction widths r_1 and r_2.

Where the concentrated load is a moving one, the critical position will be at the center of the span. The effective reaction-widths are then equal to each other and the effective width of the slab for the computation of moment of resistance is equal to either.

Since loads arising from wheels or rollers are often applied over considerably elongated areas, the orientation of the load on the span is an important consideration. For such loads, the distance d is always small—in theory only the width of a line—but the distance c may, for rollers, be as much as 48 in., and is commonly from 12 to 20 in.

The two extreme arrangements of the load, indicated in Fig. 2, will be compared. In Fig. 2 (a), the axis of the machine or vehicle is at right angles to the direction of the span of the slab, while in Fig. 2 (b), it is parallel to this direction. For the usual system of floor construction in a bridge, with the slabs supported on longitudinal stringers, the axis of the machine for the case shown in Fig. 2 (a) would be parallel to the axis of the bridge and for the case indicated in Fig. 2 (b), transverse to it. The latter is a contingency that must be considered in wide bridges. When the load is placed as shown in Fig. 2 (a), it follows from what has already been said that the effective width of the slab e_1 is given by the expression

$$e_1 = (l + c)\ tan\ \phi + d$$

and when the arrangement is that of Fig. 2(b), the effective width is

$$e_2 = (l + d)\ tan\ \phi + c$$

Since c is ordinarily very much greater than d, e_2 is greater than e_1 and the location and orientation of a wheel load most seriously stressing a slab is as shown in Fig. 2(a) and *not* as shown in Fig. 2(b).

For bearing areas differing in shape from those shown in Figs. 1 and 2, equivalent rectangles may be substituted.

When the length b of the slab is less than twice the span length l, it becomes necessary to make some reduction in the effective width. Reference has already been made to the reductions recommended by Mr. Slater. Upon comparing the effective widths indicated by the tests of wide cantilever beams resting on narrow supports, reported by Mr. Lord, with the effective widths estimated by Mr. Slater's rules, it has been shown that the latter give results well on the safe side. The application of these rules to concrete footings also gives results on the side of safety. The writer would therefore suggest that until further experimental evidence is available the values of *tan* ϕ in any calculation of effective widths, be assumed substantially in accordance with those given by Mr. Slater's graph, or as follows:

$\frac{b}{l}$	tan ϕ
0.4	0.35
0.5	0.42
0.6	0.48
0.8	0.54
1.0	0.59
1.2	0.62
1.4	0.64
1.6	0.65
1.8	0.66
2.0 and over	0.67

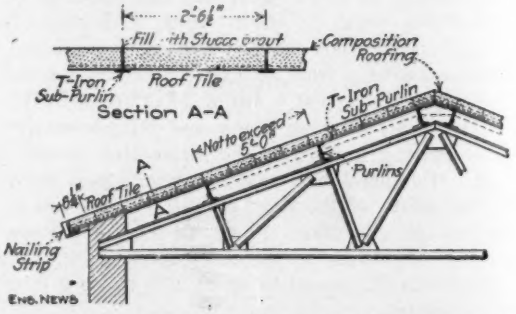
Although no experimental investigation of the relative distributing effects of various types of slab reinforcement has been made, it is probable that a form involving diagonal strands or rods is most efficient. Maximum stiffness along lines radiating from the load within the effective angle and the provision of a large number of strands directly under the load would thus be secured. Upon these considerations the delivery of the load to the supports over a considerable effective width largely depends.

The effect of a gravel cushion or other wearing surface in distributing concentrated loads over bridge floors has not been considered, as this is a matter not entering into the present discussion. Having found the area of the slab to which the load is applied by this surfacing, the problem is of precisely the same kind as arises when loads are applied directly to the slab. The only difference is that the bearing area of the concentrated load is increased with the use of a wearing surface and consequently the effective width of the slab also becomes greater.

Gypsum Slabs for Roof Construction

Rectangular gypsum slabs supported between inverted T-shapel steel sections and covered with a tar felt and gravel offer a light, insulated fireproof roof construction which has been developed by the U. S. Gypsum Co., Chicago, and used in their plant recently finished at Oakfield, N. Y. The accompanying figure shows the details of the construction.

The gypsum slab used in the "pyrobar" tile, made by this company, produced solid, without the cylindrical longitudinal openings, and reinforced, near the surface with a light wire mesh. The tile is 3x12x30 in., and weighs 13 lb. per sq ft.



TYPICAL ROOFING DETAIL SHOWING REINFORCED GYPSUM TILE LAID BETWEEN T-SECTION SUBPURLINS

Steel subpurlins of inverted T-section, 2¼x2¼ in., are used, placed 2 ft. 6½ in. c. to c. On the lower flanges of these purlins the tile are laid, starting at the lower section of the roof and fitting the slabs snugly together. The joint between the ends of adjacent tile and over the stem of the T, are filled with a plaster grout, which binds the tile together and makes a smooth upper surface ready for the application of a tar, felt and gravel roof, laid according to standard practice. The composition roof is laid directly on the gypsum tile.

Gypsum tile can be cut and fitted easily, and can be placed in between the steel purlins, using common labor practically altogether. The tile are laid directly on the purlins, and the work requires no scaffolding or false-

work. Forty tile cover one square (100 sq.ft.) of roof. The manufacturers state that such a roof will carry a safe load of 150 lb. per sq.ft., an ultimate load of 600 lb. per sq.ft.

Dump Cars for Transporting Garbage By Rail

At some cities where garbage-disposal plants are located at a distance from the city, the material is transported by rail, and a special type of dump car has been devised for this service.

One of these cars, as used at Cleveland, Ohio, is shown in Figs. 1 and 2. It is of steel construction, the underframe having two fish-belly plate-girder sills 24 in. deep at the middle. At each end are standards carrying a horizontal transverse perforated plate, the holes in which engage with studs on a cast-steel rocker on the end of the car body. The body is semi-cylindrical, made thoroughly water tight, and braced by three cross partitiions which are perforated to allow liquids to find the same level throughout the car. It is supported on the end rockers and on three intermediate trains of rolls. It is provided with double covers, hinged at the middle and clamped to the top flange angle of the body when closed.

The body is held in position by side chains having the lower end attached to the sills and the upper end to a

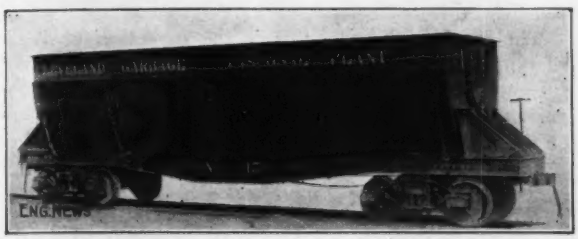


FIG. 1. DUMP CAR FOR TRANSPORTING GARBAGE AT CLEVELAND, OHIO

(Orenstein-Arthur Koppel Co., Koppel, Penn., builders.)

shelf angle on the body. These are fastened with a shackle which can be disengaged readily. To dump and right the car, a crane or air hoist is used, its hook engaging a loop on the side of the body. Very little power is required, owing to the way in which the body is supported. The car is washed out with a hose when dumped, so as to remove any material adhering to it.

The cars are built with a capacity of 1450 and 1800 cu. ft., or 40 and 50 tons. The 40-ton cars are 37 ft. 4 in. long over the end sills, 30 ft. long inside, 10 ft. 1 in. wide over all, and 10 ft. 10 in. high. They have arch-bar trucks, and are built and equipped to conform to M. C. B. standards and the requirements of the Interstate Commerce Commission. The cars are built by the Orenstein-Arthur Koppel Co., of Koppel, Penn., and we are indebted to the company for drawings and information.

In regard to the garbage transportation service by rail, the city of Cleveland, Ohio, operates 14 cars in connection with its municipal garbage-disposal plant, two of which are of 50 tons capacity. Mr. Hobbs, City Commissioner, informs us that the haul is seven miles, over the Baltimore & Ohio R.R., which furnishes the locomotive for the work. The city pays the railway 15c. per ton for the haul, and in 1913 the service averaged 170 tons per day. The Public Service Department considers this the cheapest and most satisfactory method the city has been able to find for the removal of garbage to the disposal plant.

We are informed that the Standard Reduction &

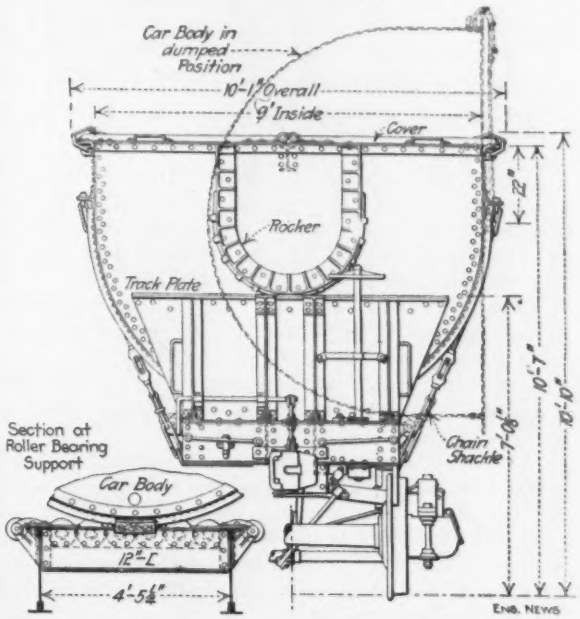


FIG. 2. DETAILS OF DUMP CAR FOR GARBAGE

Chemical Co., of St. Louis, has 18 of the cars in service; the city of Columbus, Ohio, has six, and the Garbage Reduction Co., of Toledo, has six. At Toledo, cars for this purpose were purchased by a former city administration to be used in connection with hauling garbage from a loading station centrally located. We are advised by Mr. McKechnie, Chief Engineer of the Department of Public Service, that on account of objections from property owners the loading station was never built and the cars were sold.

At Columbus, O., the garbage is collected in rear-dump wagons and hauled to a loading station, located in the central part of the city, where the wagons dump direct into the special garbage cars. The city has been operating four 40-ton dump cars and has purchased an additional 50-ton car, all of the type described above. The cost of these cars was approximately $1900 each. The garbage reduction plant is located about 3¼ miles south of the loading station. The cars are hauled two miles of this distance over the municipal railway track (built originally to serve the sewage disposal plant) and the remainder of this distance over the tracks of the Hocking Valley Ry. The railway delivers the loaded cars to the reduction plant and returns the empty cars to the loading station between 9:00 p.m. and 12:00 p.m. each day for a flat rate of $15 per trip, regardless of the number of cars handled. The above charge amounted to 25c. per ton of garbage hauled during 1912 and 21c. per ton during 1913.

The practice has been to load two cars each day. During the summer, however, the garbage averages nearly 90 tons per day and has reached a maximum of 130 tons,

so that the cars were becoming badly overloaded. The Department of Public Service now plans to use three 40-ton cars one day and a 50-ton and 40-ton car the next day. During the winter months, when the garbage amounts to about 55 tons daily, there is occasionally a day when no garbage is shipped. The average for the year, 1913 was 72 tons daily. We are informed by John C. Prior, Assistant Engineer of the Department of Public Service, that the cars have proved quite satisfactory and the arrangement is much more economical than delivering the garbage all the way by teams.

Hauling City Waste on Street Railways at Chicago

The use of dump cars on electric street-car lines for hauling street sweepings and miscellaneous waste (but not garbage) is practiced in Chicago under the direction of the Bureau of Streets (Department of Public Works). This service was instituted in February, 1911, the refuse being hauled to abandoned clay pits in the northwestern part of the city. The loading station consists of an elevated platform at 15th Place and Loomis St., with an incline at each end, so that the wagons can be driven upon it and dump their loads into the cars, which stand on tracks on both sides of the platform.

The initial equipment consisted of six wooden dump cars and a motor dump car, each with a capacity of 36 yd. Six steel dump cars and an air-dump motor car have since been added to the equipment. The steel dump cars are constructed with three V-shaped buckets having a capacity of 10-yd. each, and a canvas cover is fitted over each car when loaded.

A train consists of three cars, including the motor dump car. The city of Chicago pays the railway company $25 per day for each motor and $6 per day for each trailer used, the motor being owned by the railway company and the trailers by the city. It is the intention to establish another loading station of this kind on Goose Island in the near future, and 12 additional steel dump cars have been ordered. For information as to the above we are indebted to Walter G. Leininger, Superintendent of Streets, Chicago.

A Spring Guard-Rail for Frogs

The frog guard-rail is a troublesome feature in track maintenance, owing to the severe service imposed upon it in holding (or forcing) wheels into proper line as they pass the frog. If the guard-rail is not secured to the track rail, it will be forced away from its proper position by the continuous pressure and blows of the wheels, the flangeway being widened and the guard-rail rendered practically inoperative. If the guard-rail is bolted rigidly to the track rail, as is now general practice, the blows against it tend to pull the track out of line, and often result in the stretching and breaking of the attachments, besides causing severe wear on the frog point.

To meet these conditions, a spring-cushioned attachment has been devised, using springs which are stiff enough to hold the guard-rail in place under normal conditions but which will yield slightly under heavy blows. The bolt is set at an angle instead of horizontally, so that the shock is taken directly at the head of the guard rail and transmitted to the base of the track rail. A lateral blow on the guard-rail will result in a partial upward reaction on the track rail, and this will be resisted by the weight on the wheel which delivers the blow.

The construction is shown in Fig. 1, and Fig. 2 shows a 15-ft. guard held by four of the spring-bolt attachments. The bolt *A* is long enough to carry the spring *B*, which is set up to the desired tension by the nut and bearing plate. The head of the bolt fits into a grooved block *C*, on the guard-rail, and the spring bears upon block *D*, which has a bearing against the head, web and base of the track rail. There is the usual spacing block *E*, between the rails, bored to fit the bolt.

Fig. 1. A Frog Guard Rail with Spring Attachments

We are informed, however, that the device does not meet with the approval of the railway engineers. They consider that it is wrong in principle, in that the guard-rail yields when it is needed to protect the frog point, and that a guard-rail should be held rigidly in its normal relation to the track rail. For this reason the one or two guard-rails which are in use have the spring action restrained by bolts.

This cushioned guard-rail is the invention of E. J. Shoffner, of Roanoke, Va., and has been in use at some turnouts on the Norfolk & Western R.R. for over five years. The maintenance work has been very little, none of the bolts show any stretching, and there has been but

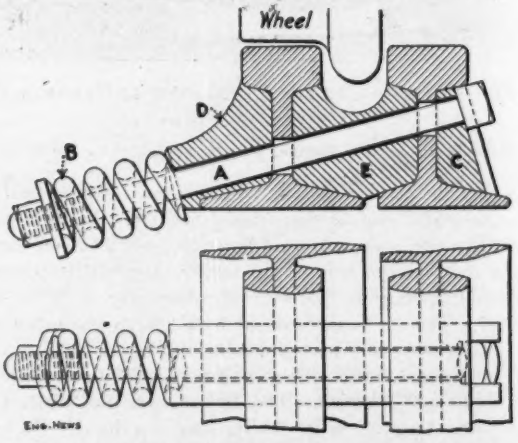


Fig. 2. A 15-Ft. Guard Rail with Four Spring Attachments

slight wear of either the guard-rail or the frog point. We are informed, however, that the device does not meet with the approval of the railway engineers. They consider that it is wrong in principle in that the guard-rail yields when it is needed to protect the frog point, and that a guard-rail should be held rigidly in its normal relation to the track rail. For this reason the few spring guard-rails which are in use have the spring action restrained by bolts. In this connection mention may be made of

the Conley frog, in which the wheel is guided by the frog itself, dispensing with the ordinary guard-rails which guide the wheel through the medium of the axle and opposite wheel.

The spring guard-rail described above is manufactured by the Virginia Railway Supply Co., 500 Dickson Building, Norfolk, Va., and the company reports trial orders from other railways.

An Electric Air-Hammer Drill

The accompanying figures show the latest design of a type of electric drill that has been under commercial development, with considerable field service, for several years. The fundamental principle involved in the mechanism has been used in a line of electric coal-punching machines, though the idea of applying it generally antedates the application to coal mining. It was planned to make the development first for coal machinery, then for rock drills and finally for riveters, pile drivers, etc.

Electricity is used as the motive power but the blow is delivered to the drill steel by an air-actuated hammer descending on a dolly pin. The motor compresses the air in the drill cylinder and this is immediately expanded, in delivering the drilling blows, so that there is little dissipation of the heat necessarily developed in compression.

The mechanical action is seen in Fig. 1; on the forward stroke of the motor-driven compressing piston, air

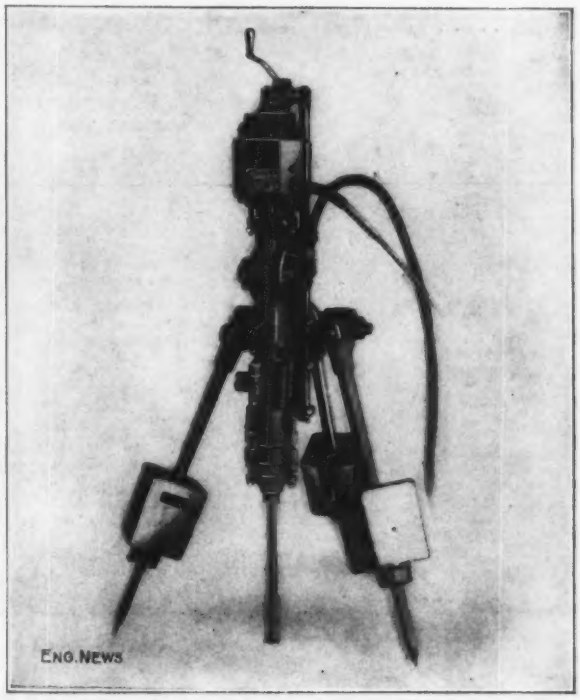


FIG. 2. ELECTRIC AIR-HAMMER DRILL AND TRIPOD MOUNTING

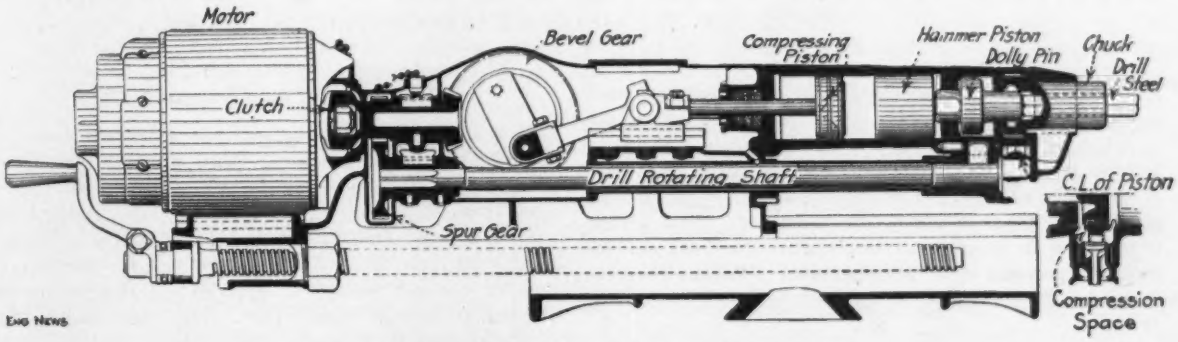


FIG. 1. SECTION THROUGH ELECTRIC AIR-HAMMER DRILL

is admitted by an inlet valve on the compression space and follows the piston. On the return stroke this air is trapped and compressed. At the same time, atmospheric pressure below the hammer piston causes this to follow the compressing piston. When the latter on its up stroke uncovers the lower row of ports the compressed air escapes between the two pistons and in expanding causes the hammer to deliver its blow. At the end of the stroke of the hammer piston, a small port is uncovered and a small amount of air allowed to escape at slightly above atmospheric pressure, so as to prevent accumulating large air pressures between pistons.

The design shown is fitted for the use of hollow drill steels and wash water continuously supplied. Water may be delivered from pressure lines if desired, but there is a small pump on the main-gear spindle and the discharge is piped to the drill-chuck chamber. The steel is rotated a fraction of a revolution after each blow, the chuck having a ratchet-and-pawl connection to a small crank arm on a long shaft running back to the motor gear, where it connects with the latter by spur pinion and gear.

Engine-indicator cards taken on these pieces of apparatus have shown an expansion curve lying close to the theoretical adiabatic line and indicating small heat losses. A maximum air compression of some 60 lb. per sq.in. is reached and, of course, heat is generated by this action the same as in any compressing machine. But the heat is absorbed by the immediate expansion and the only noticeable temperature is that of the cylinder walls resulting from absorption. This in no case exceeds 150° F. This latest type (3-in. cylinder and 3-in. stroke) is driven by a 2½-hp. motor (direct- or alternating-current as desired) and this is claimed to draw only from $\frac{1}{8}$ to $\frac{1}{10}$ the amount of energy required for a distinct supply of compressed air to an air drill.

This machine is built by the Pneumatic Machine Co., Syracuse, N. Y., under the designs of Alexander Palmros, Chief Engineer.

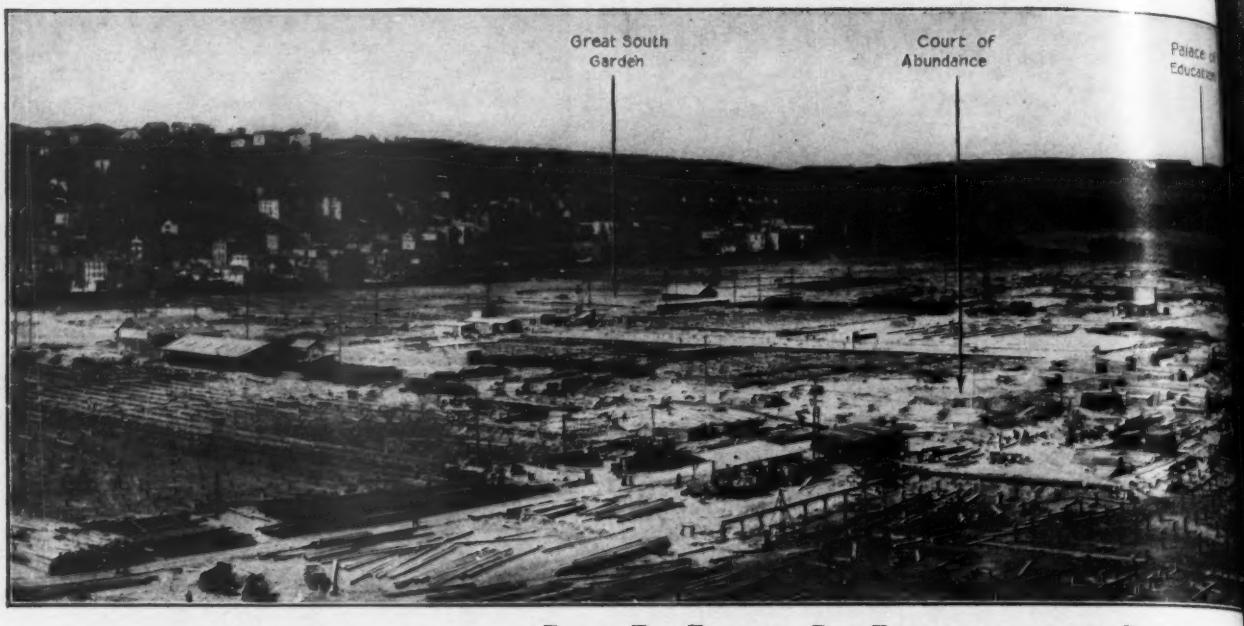


FIG. 1. THE ELABORATE PILE FOUNDATIONS FOR THE BUILDINGS OF

Soft-Ground Foundations, Panama-Pacific International Exposition

PENETRATION AND LOAD TESTS OF PILES

BY L. F. LEUREY*

SYNOPSIS—Heavy buildings to be erected on soft sand and mud—hydraulic-dredge fill. Continued subsidence of fill. Results of auger borings. Penetration and load tests of piles. Pile foundations adopted for a large part of the building area. Over 600,000 lin.ft. of pile driven at cost of 24½c. in place, 14c. delivered. This article is intended to supplement and elaborate on the articles on the Exposition by A. H. Markwart, Assistant Director of Works, ENGINEERING NEWS, *Nov. 6, and Nov. 27, '13.*

The site on which the Main Exhibit Palaces of the Exposition are located was originally a tidal flat of San Francisco Bay, with occasional deeper bights that had been dredged out for wharves and anchorage for vessels. About twenty years ago, private interests received a grant of these tidal lands and built the rock sea wall which marks the shore line on the maps herewith. A large sand hill that overlay the site of the Concessions District was graded and the excavated material deposited over a large portion of the submerged area but the work was not carried to completion, and an area of water about 80 acres in extent remained to be filled by the Exposition.

The condition of the site when the Exposition took over the ground is shown approximately by the dotted minus contours shown on Fig. (A). All of this section was submerged, the average high-tide level being about Elev. —5.5 ft. and extreme high tide reaching Elev. —4.0, reckoned from San Francisco datum as zero. The average elevation of the nonsubmerged section of the site was about +1.5.

*Engineer, Panama-Pacific International Exposition Co., San Francisco, Calif.

HYDRAULIC FILL; SUBSIDENCE*

The exposition company pumped 1,300,000 cu.yd. of fill into the submerged area. This brought the surface to Elev. —2.75 approximately. The fill material averages from 60% to 70% of sand, the remainder being mud and silt. Due to the superior weight and density of this material, it crushed its way 2 to 5 ft. into the soft ooze of the old bottom, to such extent that where the original soundings showed Elev. —15, the actual bottom of the dredger fill is nearer Elev. —20.

Due to the varied lines of flow of the dredger fill and its varying character, numerous "kidneys" or watery pockets were found in the fill. These kidneys often had skins of tight sand overlying them, but on driving in a long stake or pile their presence would be indicated by a geyser of water coming up around the pile and by the corklike action of the pile in floating up when relieved of the weight of the hammer.

*GENERAL CONTROL OF SETTLEMENT OF SITE—A. H. Markwart, Assistant Director of Works, states as follows concerning the settlement of the fill on the exposition site:

"We have taken care of settlement in the design of our sewers, water piping and roads, and do not fear slight settlement. As fills go, we have had splendid success. This results, no doubt, from the care which was taken at the time the fill was being made.

"Continuous tests were conducted in order that the sand content could be kept high. The fill is from 60% to 70% sand, and when the dredged material was too high in silt the dredgers were moved to other locations in the bay where more sand prevailed.

"Great care was also taken to expel as great an amount as possible of mud of the original bottom of the basin. This was accomplished by pumping water at times instead of sand. The water carried out through the waste-gate considerable mud in solution. Further, the discharge pipes were frequently located to the south of the basin and sand was pumped, which pushed the mud toward the waste-gate and into the bay. Occasional difficulty was encountered in this mud hardening at the gate and choking it. To overcome this, dynamite was occasionally used and sludge washed out with clear water."—Editor.

THE PANAMA-PACIFIC INTERNATIONAL EXPOSITION, SAN FRANCISCO

Shortly after the fill had been pumped in, settlement stakes were driven at various points within the filled area and levels taken at regular intervals. A typical curve of settlement is shown in Fig. 2. It will be noted that the initial settlement was rapid; this was, of course, expected. But it was not expected that the settlement shown by the latter part of curve would be so uniform and continuous.

TEST BORINGS

After the fill had taken its initial rapid settlement, borings were made at the corners of all buildings of the main group. It was found that the soil was yellow beach sand overlying soft clay, or else blue dredger sand overlying blue mud and clay. Under the sites not occupied by the dredger fill, the clay is of a soft puttylike consistency and in some cases is mixed with sand; at the bottom of the clay stratum is a layer of 1 to 5 ft. of hard green sand and clay, and then yellow hardpan. Underneath the dredger sand, the same type of clay is found, but mixed with blue mud and water, and the same layer of green sand and clay and hardpan is encountered except along the north building line fronting the Bay: here the hardpan pitched below the limit of the boring tools, but was estimated from driven piles to be about 120 ft. below the surface.

The upper yellow-sand layer extends down from surface to a minimum of —18 and a maximum depth of —30. It is a clean, coarse, yellow or gray sand of uniform character. The dredger sand is much finer and more compact, and contains about 30% of silt, but due to its fineness and liquid character it has a powerful suction action on any material driven into it.

Some of the boring results are charted in Fig. 3. The locations of all borings are platted on the map in the same figure.

PILE PENETRATION AND LOADING TESTS

The approximate character and stratification of the soil and the depth of hardpan being thus determined, it was decided to drive a number of test piles of various lengths to see what the supporting power of the ground was at different depths, because it was at once apparent that if every pile were driven to hardpan the cost would run into a very large amount of money.

The test-pile locations were selected as far as possible

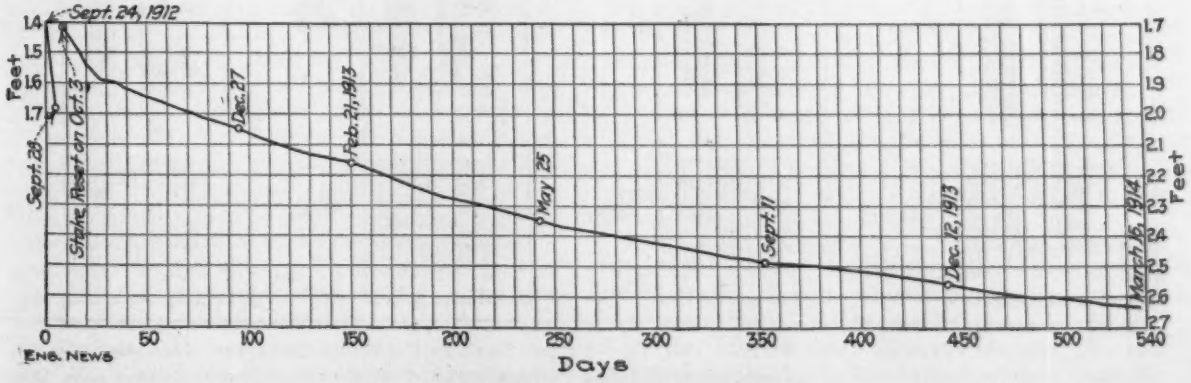


FIG. 2. SETTLEMENT DIAGRAM FOR STAKE 13

with a view to obtaining data primarily for the building itself and also for value of comparison with adjacent or similar sites in other parts of the grounds. The piles wherever possible were of the minimum size, so as to get the least favorable condition. They were loaded by means of timber sandboxes (Fig. 5) to an ultimate load at least 25% in excess of what they were expected to carry. The loads were applied gradually, in batches, and no pile was loaded further while it was in the process of settling; the aim was to get the settlement of the pile at each stage of loading.

In driving the test piles, a 2700-lb. drop hammer was used, and the fall of the hammer was maintained at 10 ft. Records were kept of the number of blows required to drive a pile for each foot of its length underground; the results are given in Fig. 6.

The Dredger-Filled Area

The buildings roughly divide themselves into two groups. The first, consisting of the Varied Industries, Manufactures, Mines and Transportation Buildings, is in the area of the new dredger fill. The second consists of the Liberal Arts, Education and Food Products Buildings, which will rest on older fill or on ground which was unsubmerged at the time the Exposition took over the land. The Machinery and Agricultural Buildings are partly within and partly without the area of the new dredger fill. The Fine Arts Building, due to its concentrated loadings and the shallow depth of hardpan under it, was not considered in the pile tests, it being apparent that the proper course was to drive piles to a hard foundation.

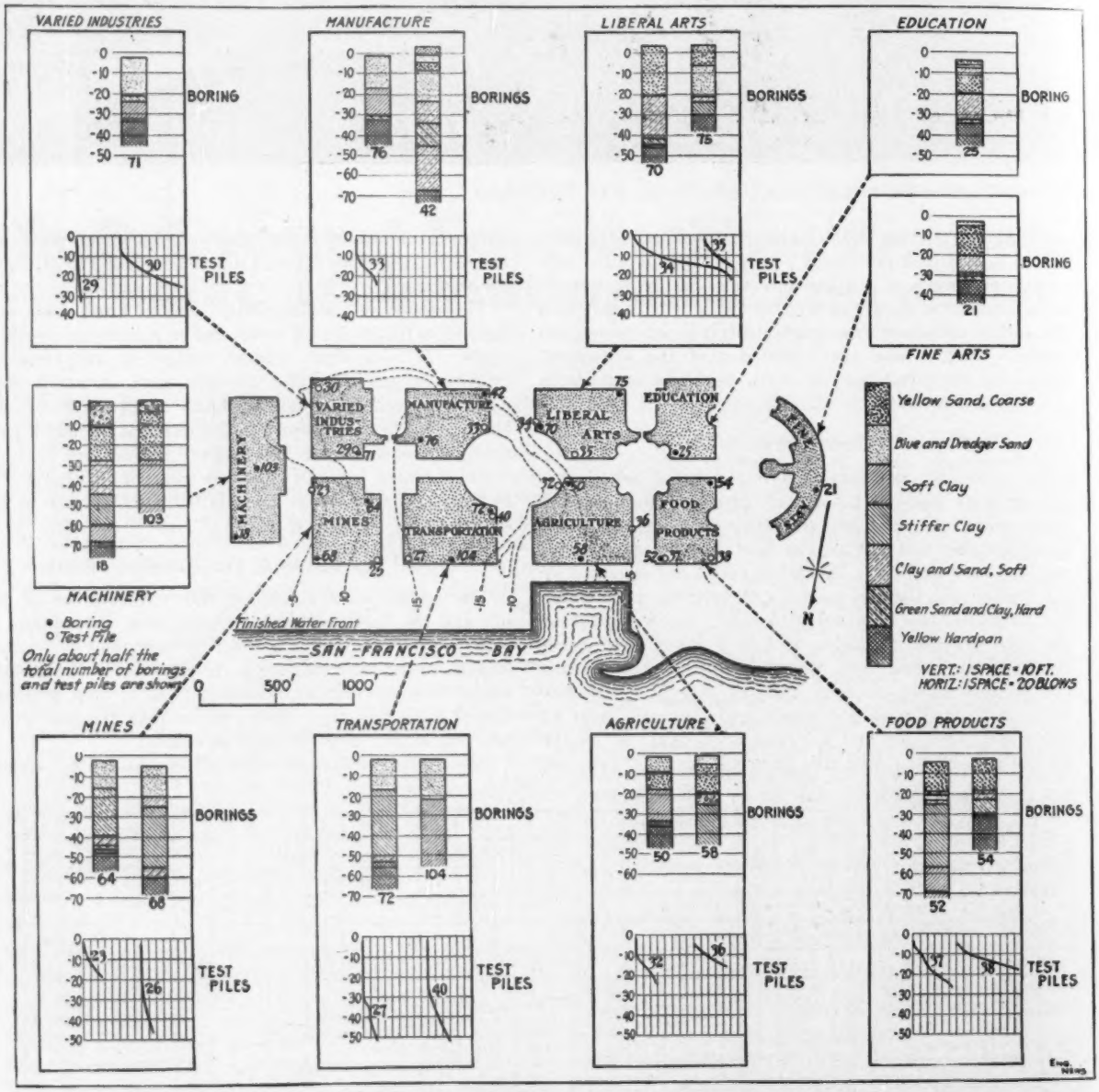


FIG. 3. TYPICAL TEST BORINGS AND TEST PILES, PANAMA-PACIFIC EXPOSITION GROUNDS

(Original condition indicated by dotted subaqueous contours. Compare Fig. 4, showing zones finally decided upon. Contours below city datum; M. H. W. is about —5.5 ft.)

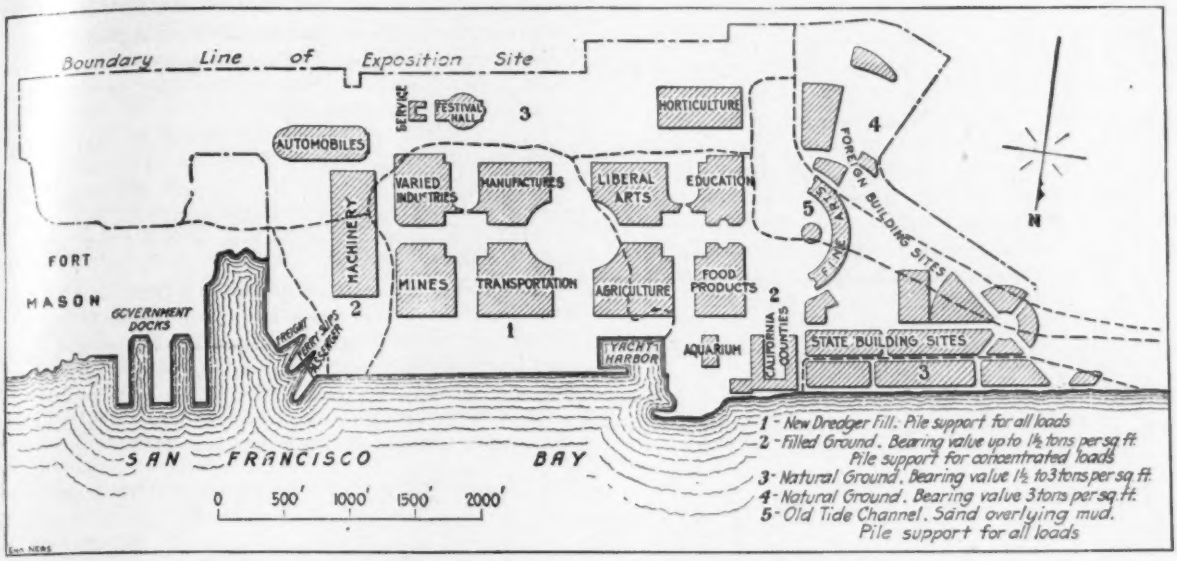


FIG. 4. ZONES OF FOUNDATION TYPES, PANAMA-PACIFIC EXPOSITION

It soon became apparent from settlement stakes and pile tests that the new dredger fill not only was non-supporting in character but was in addition *acting as a positive load on piles* driven into it or through it. After observing piles of various lengths and under various load conditions, it was deduced that, roughly, as much pile was required below the fill as there was driven through the body of the fill, to break the suction grip of the dredger sand. The question then arose how much piling was required below the fill to support the service load on each pile. Piles 45 ft. to 50 ft. below the surface seemed able to break the grip of the fill, but settled under the positive test load that was imposed on the pile.

Parallel tests were carried on by means of loaded platforms to get the supporting power of the dredger sand, and it averaged about 400 lb. per sq.ft. Of course, that figure represented the supporting capacity of the dredger sand relative to its own surface, since the whole fill was settling.

DECISION OF BOARD ON THE TRANSPORTATION BUILDING

At this juncture, the necessity arose for a definite decision as to the type of foundation to be adopted for the Transportation Building. Due to the very large amount of money involved and the necessity of feeling sure of the results to be obtained, an advisory group of consulting engineers: J. D. Galloway, C. E. Grunsky, C. Derleth, Jr., and C. B. Wing, was called in.

After studying very carefully the collected test data and witnessing the driving of four groups of piles in actual position on the building site, the board arrived at the same conclusion that the Exposition engineers had already tentatively formed, namely: That it would be unwise to attempt to carry the pile load on skin friction; and that it would be necessary to drive to a penetration of about 1 in. per blow of a No. 1 Vulcan steam hammer or its equivalent, obtaining this penetration in the hard skin of sand and clay that overlies the hardpan.

The driving of the four groups of long piles as recommended and witnessed by the consulting engineers showed that a penetration of less than ½ in. per blow could be reached with piles varying in length from 35 ft. to 75 ft., and that the intermediate ground offered practically no resistance to the penetration of the pile (in fact the weight of the hammer put all of these piles down for 35 ft. under the surface). To further confirm the latter opinion, Test Pile 28, at the southeast corner of the Transportation Building, on being loaded from 12 to 16 tons settled over 5 in. and continued to settle under a 20-ton loading until it pulled up on the hard green sand and clay—it had been originally driven to within 18 in. of this surface. It will be readily noted by referring to the penetration record, Fig. A, that this pile encountered practically no resistance to driving.

It was suggested at first to separate the floors from the building and carry them on platform foundations, but when it was considered that the relative value of the dredger fill for supporting loads was only 400 lb. per sq.ft., and that in addition some device had to be provided against the absolute settlement of the whole fill, it was finally decided to carry floors, tracks, walls and columns all on piling. The loathness of the engineers to accept what seemed inevitable, that is, long piles on this building, is explained by the fact that about 4500 piles, totaling 280,000 ft. below cutoff, were required for this building alone—for only ten months service.

The decision finally arrived at on the Transportation Building, substantiated by the tests made on the other sites within the area of the dredger fill, forced the conclusion that the same character of foundation must be obtained on these sites, and it was so decided to drive piling on the Manufactures, Varied Industries, Mines and on the affected portions of the Agriculture and Machinery Buildings into the hard, green sand and clay medium overlying the hardpan.

Eight months after driving the settlement stakes, the dredger sand was still continuing to settle at a uniform rate, amounting to an average of 1 in. per month. In places where there is a superposed fill, such as the 4-ft. fill for the Court of Honor, this rate has been increased to as much as 2½ in. per month for the first month after this fill was placed.

The Area of Original Ground

For the building sites not affected by the recent dredger fill, comparative tests showed that it was at least as good (if not better) to rest the pile within the top layer of sand as to drive it through only a few feet.

At the outset of all pile tests, it was determined to make the maximum service load of all piles 20 tons, and to determine the bearing power of short piles in the local sand formation. Six test piles were placed in the Education Building 14 ft. to 34 ft. below cutoff. All of these piles stood a load of 25 tons without settlement. The shortest pile of all, 14 ft. in the ground, was loaded to 29 tons without settlement. Seven other tests with short piles were conducted in ground of similar character (34 and 35 in Liberal Arts, 37 and 38 in Food Products, 41 and 42 in Machinery, and 36 in Agricultural Building), all of which confirmed the tests on the

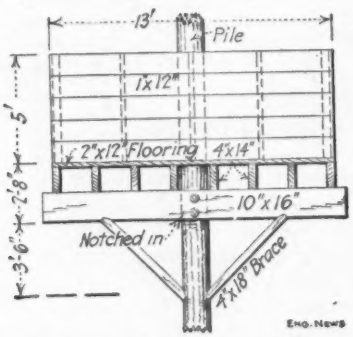


Fig. 5. Sandbox Used in Pile-Loading Tests

Education Building, namely: that short piles driven into and stopped within the sand layer of the old fill had ample factor of safety for carrying loads not exceeding 20 tons per pile.

The question might naturally be brought up that, if the sand were capable of supporting short piles, it would also be capable of supporting spread footings. Spread footings were in fact considered, assigning to the sand a supporting value of 3000 lb. safe, which figure was determined from loaded-platform tests made on the sites. Short piles, however, figured out cheaper, and possessed other advantages (simplicity of footing design, adaptability to all characters and elevations of surface), which finally led to the adoption of this type. It was thought also that pile footings in the territory of old or new fill would offer a better resistance in case of possible earthquake, as experience in the past indicates that structures were most affected when resting on filled ground by means of spread or raft footings.

The greatest observed settlement on any of these piles within the sand layer and loaded to 20 tons was 3/8 in. It was consequently decided that piles should be driven into, but not through, the sand layer, for the Education, Food Products, Liberal Arts and such portions of the Machinery and Agricultural Buildings as were of similar character, for the support of concentrated loads. The decision to follow this procedure instead of driving through the substratum of soft clay resulted in the saving of a great many feet of pile.

Foundation Zones

After a general study of the pile tests, platform tests and borings coördinated with the information derived from studying the history of the site, the Zone Map, Fig. 4, was outlined. The zone areas thus established can naturally only furnish information of approximate character, this information being substantiated by local tests at all important building sites.

In this manner, it was decided to rest the Automobile Building on spread footings, 3 tons per sq.ft. safe.

The Festival Hall and Horicultural Building Sites were assigned a value of 2½ tons per sq.ft., but it was decided to support the main dome of the Horticultural Building on piling.

The ground within the Concessions District has a high supporting value, and consequently requires no piling for the support of building structures. The States and Foreign Building Sites have a high average supporting value, but some piling is required in Zone 5.

The cost of the tests was very low, when compared to the total cost of foundation piling (see Tables I and II).

TABLE I. QUANTITIES AND COSTS OF FOUNDATION PILES, PANAMA-PACIFIC INTERNATIONAL EXPOSITION

Buildings	No. of piles driven	Lin.ft. piling driven	Lin.ft. piling cut-off	Piling below cut-off	Percentage of waste	Average length of pile below cut-off	Approx. cost per lin.ft. of pile below cut-off
Machinery..	1,577	47,573	2,487	45,086	5.2	28.6	27c.
Education..	634	11,595	1,608	9,987	13.9	15.7	40
Manufactures.....	1,591	62,292	5,429	56,863	8.7	35.7	25
Agriculture	1,374	59,710	2,253	57,457	3.8	41.8	23½
Food Products	665	12,015	2,388	9,627	19.9	14.5	40
Liberal Arts	751	10,989	1,241	9,747	11.4	13	40
Transportation	4,541	298,674	11,849	286,915	4.0	63.2	23
Fine Arts	1,051	31,111	4,173	26,938	13.4	25.6	25
Varied Industries	1,444	52,367	2,756	49,611	5.2	34.3	22
Mines	2,026	98,463	4,609	93,461	4.6	46.1	22
Total	15,654			645,692		41.2	24½c.

Note:—All piles were driven with No. 1 Vulcan Steam Hammers.

Weight of moving element approximately, lb.	5000
Fall of hammer element approximately, ft.	3½
Average cost of Douglas fir piles delivered at site, per lin.ft.	14c
Approximate cost of erecting and dismantling pile driver	$500

TABLE II. UNIT-COSTS OF TESTS

(Total cost, including material, labor and engineering.)

Loading piles by means of 30-ton sandboxes, as shown in sketch, per test	$70
Loading platforms with sand load, per test	15
Hand borings with auger and 3-in. casing, holes 15 to 65 ft. deep in sand and clay, per lin.ft.	0.26

The results obtained were not only valuable in giving the engineers assurance in the use of short piling, but also reduced contractors' prices by furnishing them with accurate information, thus eliminating unnecessary waste.

Grasses for Holding Earth Banks in Place—In maintaining earth fills or on the sides of cuts, hardy well rooted low-growing grasses offer probably the best protection against erosion. In a recent issue of the "Service Bulletin" published monthly by the Iowa State Highway Commission, Ames, Ia., Prof. L. H. Pammel states that of the grasses developing a good system of root the following are the best: Hungarian brome grass (Bromus inermis), hairy brome grass (Bromus marginatus), Canadian blue grass (Poa compressa), blue grass (Poa pratensis), fescue grass (Festuca pratensis), western wheat grass (Agropyron Smithii). Quack grass produces good roots but its use is not to be recommended because it spreads freely to the fields.

One difficulty with the Hungarian brome grass is that it is not so good on the south or sunny slope of the hills because the hot rays of the sun interfere with the germination and growth of the grass. On the sunny slope western wheat grass or Canadian blue grass can well be used.

Planting on cut sides is another problem, and experience seems to indicate that some of the legumes will succeed better than the grasses, as it is always difficult to get the grasses to start. Sweet clover or creeping honeysuckle is very good for the sides of cuts.

Experiments have shown that all the vegetation of this type is easily killed by cultivation and that there is little fear of its spreading to the fields.

Tests of the Resistance to Corrosion of Various Brands of Iron and Steel

The Water Service Committee of the American Railway Engineering Association has been engaged for two years past on a series of tests to determine the relative resistance to corrosion of various samples of iron and steel. The results of these tests are of especial interest in view of the widely published claims concerning the resistance to corrosion of certain brands of iron and steel of special chemical composition.

The tests have been carried on under direction of J. L. Campbell, Engineer of Maintenance-of-Way, of the El Paso & Southwestern Ry., who is Vice-Chairman of the Water Service Committee. A report of these tests was presented at the annual meeting of the American Railway Engineering Association in March, and was briefly summarized in our report of that meeting. Since that time, the tests have been continued so that the results are now available from a twelve-month exposure of the various samples. These results, as will be seen by reference to the tables, indicate that corrosion proceeds at a substantially uniform rate on all kinds of rolled iron and steel, no matter what its chemical composition. We print the report below, as follows:

The resistance to corrosion of iron and steel plates has been the subject of considerable study and discussion by the Water Service Committee for several years with the view to determining the most suitable and most lasting material for steel water tanks. Following is a report prepared by J. L. Campbell, Vice-Chairman of the Water Service Committee, on a series of tests of various metals conducted by him, outlining their relative resistance to corrosive influences:

In the tests beginning May 1, 1913, seven samples of iron and steel were selected, as follows:

No. 1, Charcoal iron.

No. 2, Openhearth steel.

No. 3, Openhearth steel containing 0.40 per cent. of copper.

No. 4, Openhearth steel containing 1.00 per cent. of copper.

No. 5, Ingot iron.

No. 6, same as No. 3, except as to preliminary surface preparation as hereinafter described.

No. 7, same as No. 4, except as to preliminary surface preparation as hereinafter described.

Each sample contained four pieces. The samples were each ⅛ in. thick, 2 in. wide and 2 to 3 in. long.

In the following analysis, the figures in the first column are as given by the manufacturers, and in the second column, as given by S. W. Parr, Professor of Applied Chemistry, under the direction of A. N. Talbot, Professor of Civil Engineering, University of Illinois.

	Chemical Analysis by: Makers	Parr
No. 1, Charcoal iron.		Per cent.
Carbon		0.041
Manganese		0.205
Phosphorus	No	0.049
Sulphur	analysis	0.032
Copper		0.00
Silicon		0.033
No. 2, Openhearth steel manufactured by the Carnegie Steel Co.	Per cent.	Per cent.
Carbon	0.15	0.144
Manganese	.037	0.394
Phosphorus	0.037	0.038
Sulphur	0.037	0.039
Copper	Trace	0.00
Silicon	0.00	0.036
No. 3, Openhearth steel manufactured by the Carnegie Steel Co.		
Carbon	0.12	0.141
Manganese	0.38	0.418
Phosphorus	0.020	0.037
Sulphur	0.032	0.028
Copper	0.40	0.43
Silicon	0.00	0.021
No. 4, Openhearth steel manufactured by the Carnegie Steel Co.		
Carbon	0.15	0.139
Manganese	0.45	0.492
Phosphorus	0.023	0.038
Sulphur	0.033	0.034
Copper	1.00	0.98
Silicon	0.00	0.033
No. 5, Ingot iron manufactured by the American Rolling Mill Co.		
Carbon	0.012	0.030
Manganese	0.025	0.180
Phosphorus	0.006	0.017
Sulphur	0.028	0.056
Copper	0.042	0.00
Oxygen	0.035	0.00
Nitrogen	0.004	0.00
Silicon	Trace	0.014

No. 6, Quality, manufacture and analysis the same as No. 3.

No. 7, Quality, manufacture and analysis the same as No. 4.

Five corrosive mediums were selected as follows:

No. 1, Clean sand.

No. 2, Clay soil, to which 5% of salt by weight was added.

No. 3, A mixture of equal parts of white and black alkali soils.

No. 4, Bituminous coal cinders.

No. 5, Cooling water in the overflow tank from the furnace water jackets of the Copper Queen Consolidated Mining Co., Douglas, Ariz.

Analyses of these corrosive mediums are as follows:

No. 1 Sand.

	Per cent.
Silica	82.31
Oxide of aluminum	9.44
Oxide of iron	2.98
Calcium oxide	1.86
Magnesium oxide	0.45
Oxides of sodium and potassium	3.18
Oxide of manganese	Trace
	100.22

No. 2, Clay soil plus 5 per cent. salt.

	Per cent.
Water	5.89
Silica	54.34
Oxide of aluminum	14.33
Oxide of iron	3.71
Calcium oxide	5.10
Magnesium oxide	2.15
Oxides of sodium and potassium	10.17
Oxide of manganese	0.11
Sulphuric acid, combined	0.95
Chlorine, combined	2.83
Carbonic acid, combined	2.05
Phosphoric acid, combined	0.16
	101.79

No. 3, White and black alkali soil.

	Per cent.
Water	5.10
Silica	48.46
Oxide of aluminum	8.76
Oxide of iron	2.00
Calcium oxide	9.45
Magnesium oxide	1.37
Oxides of sodium and potassium	11.10
Oxide of manganese	0.04
Sulphuric acid, combined	9.64
Chlorine, combined	3.64
Carbonic acid, combined	2.26
Phosphoric acid, combined	0.06
	101.88

No. 4, Cinders.

	Per cent.
Volatile matter	5.44
Fixed combustion carbon	24.60
Silica	41.06
Oxide of aluminum	24.76
Oxide of iron	1.54
Calcium oxide	1.60
Magnesium oxide	0.42
Oxides of sodium and potassium	1.22
Oxide of manganese	0.02
Sulphuric acid, combined	0.50
Chlorine, combined	Trace
Phosphoric acid, combined	0.07
	101.23

No. 5, Water in settling tank.

	Per cent.
Alkalinity in $CaCO_3$ parts in 100,000	21.00
Hardness in grains per U. S. gallon	3.50
Total solids	57.88
Silica	1.17
Iron oxide and alumina	0.47
Calcium carbonate	0.40
Magnesium carbonate	0.42
Sodium carbonate	14.00
Sodium sulphate	16.80
Sodium chloride	25.20

The sand, clay soil, alkali soil and the cinders were placed in aluminum pans, each pan being 4 in. deep, 11 in. wide and 16. in. long in the clear. The pans are numbered 1, 2, 3 and 4 and the settling tank at Douglas, No. 5, corresponding to the numbers designating the corrosive mediums contained by the pans. These pans are kept on the balcony of one of the south windows of the seventh floor of the office building of the El Paso & Southwestern R.R. Co., El Paso, Tex. They are exposed to sunlight and air, and the materials in them are periodically saturated with water, thereby being alternately wet

and comparatively dry, duplicating in a general way service conditions of iron and steel buried in the ground with a light covering.

In each pan, one piece of each sample of iron and steel, No. 1 to No. 7, inclusive, is placed, with the exception of pan No. 5 at Douglas, in which samples Nos. 6 and 7 were not placed. The pieces are buried in the corrosive medium by forcing them down edgeways to the middle of the depth of the pan, the pieces standing side by side about two inches apart. As this places one piece of each sample of iron or steel in each pan, it makes the corrosive test strictly comparable throughout for all the corrosive mediums.

Each piece of each sample of iron or steel was prepared for beginning the test by carefully filing off all surface oxidation until only clean, bright metal showed over all surfaces of the pieces, including the edges, with the exception of samples Nos. 6 and 7. The original surfaces of samples Nos. 3, 4, 6 and 7, containing copper, were covered with a distinct copper-colored oxidation:. The oxidation was left on Nos. 6 and 7 to determine its effect in resisting corrosion, as compared with samples Nos. 3 and 4, from which the surface oxidation was completely removed before beginning the tests.

After being prepared as above, each piece of each sample was weighed on a metric scale measuring to one centigram. All pieces of all samples were then immediately immersed in the corrosive mediums as described. At the beginning of the test, it was decided to clean and weigh the samples at the end of each three months' period, and the loss in weight in grams per square inch of exposed surface was chosen as the unit for corrosive comparison.

It has been found that the corroded samples may be thoroughly cleaned of the corrosive action, leaving the clean metal, by immersing the samples in a 10% solution of ammonium citrate, and this method of cleaning has been adopted. In this connection, and at the first cleaning, a clean piece of uncorroded iron was also immersed in the solution in order to determine if the latter itself would produce any loss of weight in the metal. Subsequent weighing of this control piece shows that there is no such loss, and the ammonium citrate appears to be a satisfactory medium for cleaning.

In the following tables, the corrosion is measured in loss of weight in grams per square inch of exposed surface and edges, as follows:

PAN NO. 1, CLEAN SAND

Sample No.		Loss in grams per sq.in. of exposed surfaces and edges In 3 mo.	In 6 mo.	In 9 mo.	In 12 mo.
1	Charcoal iron	0.41	0.96	1.35	1.73
2	Carnegie plain O. H. steel	0.42	0.86	1.21	1.56
3	Carnegie 0.4% copper O. H. steel	0.45	0.87	1.24	1.66
4	Carnegie 1.0% copper O. H. steel	0.45	0.90	1.25	1.61
5	Ingot iron	0.43	0.87	1.23	1.61
6	Same as No. 3 not filed	0.34	0.90	1.28	1.61
7	Same as No. 4 not filed	0.31	0.84	1.21	1.58

Tests begun May 1, 1913, in office of Engr. M. of W., E. P. & S. W. R.R. Co.

PAN NO. 2, CLAY SOIL +5% SALT BY WEIGHT

Sample No.		Loss in grams per sq.in. of exposed surfaces and edges In 3 mo.	In 6 mo.	In 9 mo.	In 12 mo.
1	Charcoal iron	0.13	0.34	0.51	0.85
2	Carnegie plain O. H. steel	0.14	0.32	0.53	0.72
3	Carnegie 0.4% copper O. H. steel	0.20	0.44	0.71	0.95
4	Carnegie 1.0% copper O. H. steel	0.14	0.34	0.60	0.84
5	Ingot iron	0.16	0.41	0.66	0.87
6	Same as No. 3 not filed	0.13	0.35	0.66	0.86
7	Same as No. 4 not filed	0.14	0.32	0.62	0.85

Tests begun May 1, 1913, in office of Engr. M. of W., E. P. & S. W. R.R. Co.

PAN NO. 3, WHITE AND BLACK ALKALI SOILS

Sample No.		Loss in grams per sq.in. of exposed surfaces and edges In 3 mo.	In 6 mo.	In 9 mo.	In 12 mo.
1	Charcoal iron	0.06	0.13	0.15	0.20
2	Carnegie plain O. H. steel	0.06	0.14	0.16	0.20
3	Carnegie 0.4% copper O. H. steel	0.06	0.14	0.17	0.22
4	Carnegie 1.0% copper O. H. steel	0.06	0.15	0.17	0.22
5	Ingot iron	0.07	0.16	0.18	0.23
6	Same as No. 3 not filed	0.04	0.12	0.18	0.23
7	Same as No. 4 not filed	0.04	0.10	0.15	0.23

Tests begun May 1, 1913, in office of Engr. M. of W., E. P. & S. W. R.R. Co.

PAN NO. 4, CINDERS

Sample No.		Loss in grams per sq.in. of exposed surfaces and edges In 3 mo.	In 6 mo.	In 9 mo.	In 12 mo.
1	Charcoal iron	0.76	1.23	1.64	2.00
2	Carnegie plain O. H. steel	0.89	1.38	1.73	2.09
3	Carnegie 0.4% copper O. H. steel	0.78	1.34	1.70	2.04
4	Carnegie 1.0% copper O. H. steel	0.78	1.21	1.59	1.93
5	Ingot iron	0.71	1.23	1.60	1.95
6	Same as No. 3 not filed	0.43	0.93	1.29	1.72
7	Same as No. 4 not filed	0.58	1.18	1.59	1.89

Tests begun May 1, 1913, in office of Engr. M. of W., E. P. & S. W. R.R. Co.

PAN NO. 5, WATER IN OVERFLOW TANK FROM WATER JACKETS

Sample No.		Loss in grams per sq.in. of exposed surfaces and edges In 3 mo.	In 6 mo.	In 9 mo.	In 12 mo.
1	Charcoal iron	1.68	2.188	2.83	[illegible]
2	Carnegie plain O. H. steel	1.62	2.172	2.76	[illegible]
3	Carnegie 0.4% copper O. H. steel	1.82	2.310	3.06	[illegible]
4	Carnegie 1.0% copper O. H. steel	1.77	2.255	3.02	[illegible]
5	Ingot iron	1.39	1.880	2.65	[illegible]
6	Same as No. 3 not filed	....	.....	....	[illegible]
7	Same as No. 4 not filed	....	.....	....	[illegible]

Your attention is directed to results on samples Nos. 6 and 7 in pans Nos. 1 to 4, inclusive, from which it appears that the copper oxidation on the surface, as compared with the identical samples Nos. 3 and 4, but with the oxidation removed from the latter, protected samples Nos. 6 and 7 for three months, but this protection appears to have disappeared at the end of six months.

Perhaps the most significant figures are those showing the relative corrosion of samples Nos. 2, 3 and 4, as they are of the same grade of steel, made by the same manufacturer, and are presumably identical in quality, the difference in copper content excepted. In regard to the addition of the copper, we quote from the letter of the manufacturer, as follows:

In regard to the addition of copper, would say that the heats are made up as far as possible with copper scrap and any deficiency in the copper content is made up by adding the requisite amount of metallic copper to the bath in the open-hearth furnace about fifteen minutes or half an hour before tapping. The copper, therefore, has ample opportunity to become evenly distributed in the steel, particularly by the mixing action which takes place when the steel runs from the surface into the ladle.

It will be observed that the corrosion of the ingot iron is substantially the same as the other samples, except in pan No. 5, containing the water from the overflow tank of the furnace water jackets at Douglas, Ariz. Of this water, Mr. Stuart W. French, General Manager, Copper Queen Consolidated Mining Company, says: "We have found that the water is extremely corrosive in our water jackets up to a temperature of say, 150° F. Above that temperature it seems to have little action. It is good water for our boilers, but in all cold-water pipes and water jackets, where the water is more or less cool, pitting action is very strong."

The analysis of this water is given above. The steel water jackets of the furnaces mentioned require frequent renewal on account of the corrosion specified by Mr. French. It was for this reason that this water was also selected as one of the corrosive mediums in this test. The corrosive action of this water appears to be somewhat similar to that of an acid, and it will be observed that, while the corrosion of the ingot iron in the other corrosive mediums is not materially different from the other samples, it shows considerably less corrosion in the water, which conforms with its known ability to resist the sulphuric-acid test.

The corrosion of all samples in the clean sand is greater than in the clay and alkali soils. Rather the reverse would be expected, especially in the alkali soil. This may be partially due to the fact that, while the sand is porous and allows a comparatively free circulation of air, the clay and alkali soils are very close-grained and practically exclude the air.

Tar Treatment of Macadam Roads in Lytham, England—Tar painting has been carried out in Lytham for the past nine years, this town being probably the first in the North of England to adopt it. Some 300 sq.yd. of the road along the sea front, which was an ordinary macadam road, were tried experimentally, half with limestone and half with granite chippings. The portion laid with limestone was for a time almost as dusty as ordinary macadam, but the granite, being a harder material than the limestone, was practically dustless when put on, and nothing but granite has since been used for tar-painted roads. When tar painting a new road an ordinary 160-gal. tar boiler is used, which is convertible into a sprinkler with gas piping perforated to form a spray; but for repairs a small 80-gal. boiler fitted with a pump and movable spray is chiefly used. A distilled tar is used for all macadam work, and for tar painting 1 part distilled tar to 1½ of crude tar. The tar macadam is better if it can be "hogged" for some time before laying, but in winter, on grounds of economy and convenience in handling, as a rule, tar is put on the roads the same day that it is mixed. The tar-macadam used is composed of stone from 2 in. down to ¼ in., and this aggregate has been used long before a well known firm of contractors patented this method. [A. J. Price, Surveyor of Lytham, in the "Contract Journal," July 1.]

Editorials

The studies and experiments on which the design of the Panama-Pacific foundations was based, are probably as extensive a set of soil and pile tests as has ever been made. They testify to the prudence and skill of the engineers in charge of the exposition. Sound construction and maximum economy were secured. Erecting the great Exposition buildings on the yielding surface of a new sand-and-mud fill was in every respect a formidable task.

These tests are among the landmarks of a new spirit in foundation work. The future is to be a time when tests—both experimental tests and proof tests—will be the regular thing in foundation practice. The most advanced engineers have already come to this point. To guess at the bearing power of soil, or to "determine" the capacity of a pile by a mathematical formula, is a crude way to reach safe and economical results.

An Ancient Error

Heavy indeed are the responsibilities of a technical editor! A correspondent this week calls our attention to an error in a formula published in our issue of Nov. 17, 1892—22 yr. ago! The formula was the famous ENGINEERING NEWS piledriving formula, originally developed by the late A. M. Wellington in ENGINEERING NEWS, of Dec. 29, 1888. Discussing a formula for piles driven with steam hammers in the later issue referred to, Mr. Wellington gave the formula as

$$L = \frac{2WH}{s + 0.1}$$

and wrote:

The same results will be reached if we retain the formula of par. 8, but let h = the total fall in feet in 10 blows, and S = the total set in inches in 10 blows.

Unfortunately, Mr. Wellington's handwriting was not of the plainest. His "S" looked more like a figure 5 than an S and the compositor set it up as 5. The error was undiscovered by the proofreaders and thus was handed down to posterity.

Fortunately, the remainder of the article made the error so obvious that probably none of the many engineers who have studied that editorial discussion have been seriously misled by the error. It is, however, an excellent illustration of the extent of the long continuing responsibility that the error of an editor, or even of a compositor, may make trouble long after those originally responsible for the error have passed away.

Signs to Mark Street-Car Stopping Points

In these times, when the managers of public-utility companies realize as never before the importance of cultivating the good will of their patrons, it seems worth while to again call attention to the system in use in German cities of plainly marking the places where street-railway cars stop.

In American cities, it is customary for street-railway cars to stop at or near every street corner to take on or leave passengers, but it is a constant bone of contention as to whether the stop should be on the near side or the far side of the street, and whether directly at the street crossing or beyond it, and how far beyond it.

Not only does the practice as to near-side or far-side stop vary in different cities, but it varies in the same city at different crossings. Furthermore, every little while a new city government or new management of the railway company upsets established customs and habits and changes the stop from the near side to the far side, or *vice versa*. The net result is that it is always a matter of doubt where a street car is going to stop. There are short streets and alleys in most of the older cities where a car may stop and may not.

Now when a man or woman stands on the street corner, often after having waited some time, makes frantic signals to a street-railway car to stop and then sees the car speed past, the motorman and conductor either paying no attention whatever or waving with a more or less derisive gesture at the other corner or some other place—when an incident of this sort occurs (and it probably occurs several million times every year in American cities) the would-be passenger, unless he is a person of most unusual good nature, is filled with indignation. In most cases he is ready to curse the street-railway company and all its works and workmen. What is particularly aggravating is that every such occurrence is supposed to testify to the ignorance or stupidity of the would-be passenger and he resents this as injustice. Henceforth he has it in for the street-railway company whenever he can get back at it with a vote at the polls or in the jury box. The chances are that he will vent a part of his disgust and irritation upon the motormen and conductors thereby adding a quite unnecessary increment to their contempt for the traveling public.

It would be easy to eliminate all this waste of human energy in useless anger and friction by adopting the simple plan common in every German city of having plain, neat, metal signs placed along the streets locating the points at which street-railway cars stop.

It would of course be impossible in old world cities, with their heterogeneous street arrangements to stop street cars at every crossing. Nor is there anything unreasonable in requiring a street-car passenger to walk three or four hundred feet along the street to a stopping point. He will be more than compensated by the greater rapidity of his journey once he is on the car. American street cars, in fact, stop too frequently for either the public convenience or the company's profit. With stopping points 800 ft. apart, the maximum distance a passenger has to walk along the street to a station is only 400 ft., and the maximum number of car stops per mile is only 6½.

Increasing the average speed of a street car increases its daily mileage and earnings. This is only one of the many directions in which American street-railway managers might profitably study foreign practice.

Questionable Taste in Publicity

From a recent trade publication we clip the following paragraph:

> Livingston County, Ill., had a unique celebration on Good Roads Day, Apr. 15, 1914. It was a mock burial of inferior culvert materials.
>
> A concrete monument, typifying the material and permanence of the concrete culvert, which will be used in future work, now stands above three graves in which were buried three old sections of culvert, types which are no longer considered serviceable.
>
> While this mock burial was only a bit of fun indulged in by the County Highway men, yet it had a real meaning for the taxpayers. It meant that no more of their funds are to be expended in such a way as to require constantly recurring repair charges.

This "bit of fun" is similar to that indulged in by a prominent cement company at the last Cement Show, where one booth was given over to the funeral trappings of many satin-lined and glass-topped "coffins" in which were displayed cement sacks ruined, or to continue the morbid metaphor, "killed" by various forms of bad practice.

It seems to us that it takes a most peculiar sort of mind to see anything funny in death and particularly in the depressing accompaniments of modern burial.

Cleveland's New Municipal Electric Plant: Selling Electricity at a Three-Cent Maximum Rate

Cleveland has become famous as the only city in the country where a three-cent street-railway fare prevails. It is shortly to become equally noted as the only city in the United States—or anywhere else for that matter as far as we are aware—where a municipal electric plant furnishes electric current at a *maximum* rate of 3c. per kw.-hr.

Ever since the beginning of the electric-lighting industry, the rate of 10c. a kw.-hr. for current has had something of the dignity and standing that the five-cent fare has had in street-car service. While a higher rate has prevailed and still prevails in many places, there are very few places where a lower rate has been put in force.

In the early years of electric lighting, it is probable that a good share of the companies lost money at the 10-cent rate. The equipment was inefficient and the load factor was very low, when almost the sole use of the current was for lighting. Further, the amount of current used for lighting was very small compared with what present customs require.

In the last dozen years, however, great changes have come in the electric-lighting industry. We need not here catalog them further than to say that their net result has been to cut down very greatly the cost of generating and distributing electric current when it is done on a very large scale. Of equal or greater importance than the reduction in the cost of making and distributing current has been the growth in the market for it, and especially in the market for current for operating motors and heating appliances, which are in use during hours of the day when the station would otherwise be run on very light load.

These changes have been recognized by the electric-lighting companies and they have in almost all cases put in force heavy reductions in rates to *large* consumers of electric current. A sliding scale for the sale of current prevails almost everywhere; but the old price of 10c. is the base price in most general use and is the rate which the individual householder is obliged to pay for his electric light.

Now comes the City of Cleveland with the proposal not only to sell the individual householder electric current at 3c., but to make sweeping reductions from this base rate to the larger consumers, so that manufacturers and others who wish very large quantities of current for motor service can buy it as low as 1c. per kw.-hr.

No wonder that Cleveland and its city government are being cursed throughout the electric-lighting industry! For if Cleveland shall succeed in demonstrating that the city itself can make and sell electric current at such a low rate, it will give an enormous boom to municipal ownership of electric-lighting plants. Besides this, the success of such a demonstration would inevitably compel a reduction in rates for electric current sold by the companies not only in Cleveland but the country over. And it will have to be a sweeping reduction.

The question whether the Cleveland municipal electric plant can sell current at the proposed rates and make a fair profit, therefore, is not a merely local matter, of interest to Cleveland only. It is of interest to cities and city officers the country over and to engineers who have to do with municipal work and with electric lighting and power development.

Of course the final answer to this question must be given by the test of experience. Cleveland's new municipal electric station has just begun operation and six months or a year hence figures for its operating expenses will be available. A year or two years or even longer, however, will probably pass before the station load will be fully enough developed to enable high economy to be realized; and for the full realization of the changes to grow out of the Cleveland experiment because of the increased use of electric current, resulting from the low prices, one must wait probably years longer.

But without waiting for the final test of experience, it is of interest at this time, to ascertain the reasons why the Cleveland city officials in charge of the new municipal plant believe that they can carry out their great experiment with financial success.

It will, of course, be agreed by all that a municipal electric plant—or any other municipal enterprise for that matter—should be run on a business basis. That is, it should pay its way and pay the whole of its way, and not be a burden on the taxpayers. Rates for electric current should be such as will pay not only all the expenses of operating and maintaining the plant, but a sufficient sum in addition to cover the interest on its cost and an annual allowance for depreciation which will cover wear and tear and all other conditions, and when the plant is finally worn out or obsolete should be sufficient to replace it or to pay off the bonds issued for its construction.

If any municipal enterprise falls short of this, the deficiency must in the end be made good at the expense of the taxpayers.

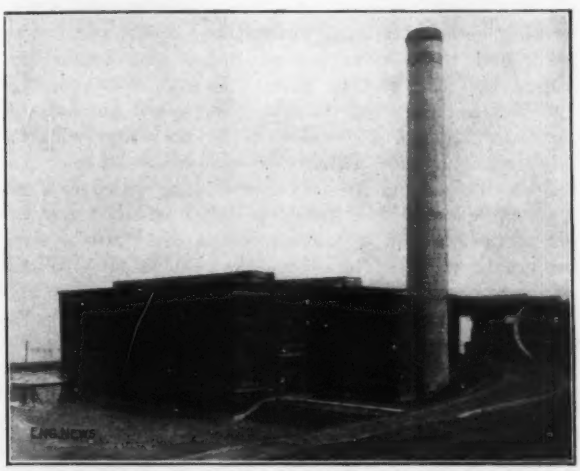
It should be understood in the first place that the municipal electric-lighting business is not a wholly new experiment for the city of Cleveland. The city in fact fell heir to two complete municipal electric-lighting plants when

it absorbed two smaller municipalities a few years ago, the outlying suburbs known as Brooklyn and Collinwood. The Collinwood plant may be dismissed without further mention, as it is of very small size, so small that its operating cost per unit of power output is very high. The Brooklyn plant, however, has been in operation for a dozen years. Begun in a modest way in 1902 from the proceeds of a bond issue of $30,000, it has been added to year by year until its book value at the present time, including, of course, both the station and the distribution system, is $435,000 in round figures.

This book value has been reached by the rather uncertain process of adding to the original cost of the plant the sums spent on its enlargement and extension and deducting about 20% for depreciation. We are, however, informed that if the whole plant were appraised on the basis of what it would cost to replace it, including land, buildings, machinery, distribution system and all, the result would probably equal the book value above quoted.

Further than this, if the Brooklyn plant were appraised on the basis of its earning power—a basis so generally favored by the electric-lighting companies for their own plants—the fairness of the book value of $435,000 above quoted would be sustained.

The Brooklyn plant in 1913 sold current of a total value of $185,698. Its total operating and maintenance expenses were $116,719, leaving net earnings of $68,979.



THE NEW MUNICIPAL ELECTRIC STATION OF CLEVELAND, OHIO

It will be seen that these net earnings were equivalent to nearly 16% upon the book value above quoted of $435,000.

This Brooklyn plant, it should be especially noted, did this business and made this handsome profit at rates which, though not so low as those soon to be put in force by the new Cleveland plant, are phenomenally low compared with those usually charged by electric-lighting companies. *The average rate for all current sold by the Brooklyn plant last year was only 3.28 cents.*

The Brooklyn generating plant, however, is to be discontinued when the new municipal plant is in operation, for the cost of generating current will be so much lower there that it will no longer be profitable to operate the Brooklyn plant.

The abandonment of the Brooklyn plant is not expected to involve any considerable loss, for the distribution network and substations will be used as before, part of the machinery will be transferred to the new municipal station and the real estate will be salable.

And now let us take up the question, what will it cost Cleveland to generate and distribute current from the new municipal plant?

It will be admitted in the first place, of course, that the cost of current at the station busbars can be predicted in advance very closely. It is well known that electric central stations of large size using moderate-cost coal are today generally running at a total operating cost of less than ½c. per kw.-hr. of current output and often considerably below this figure.

The new Cleveland municipal electric plant is to begin operations with three turbine-generator units, each of 7500-kw. capacity. An additional unit from the Brooklyn station increases the total station capacity to 25,000-kw. It receives a supply of condensing water from one of the city pumping stations close by. Its machinery equipment represents the latest practice in steam boilers, turbines, electrical equipment and auxiliaries.

A full description of the equipment of this new station, which comprises many novel features in steam-power plant practice, is to be given at the December meeting of the American Society of Mechanical Engineers by Frederick W. Ballard, Commissioner of the Cleveland Municipal lighting department and engineer in charge of construction of the new plant. A detailed description of the plant is withheld until that time.

We can testify, however, that the station design and equipment is such that its operating cost should be at least as low as that of other first-class central stations. Large boilers, automatic stokers and cheap coal supply are among the important factors in keeping down the cost. It will certainly be on the safe side to assume that the total cost of current at the station busbars when the station is fairly loaded, including all outlay for operation and maintenance will not exceed 4 mills per kw.-hr.*

How much should be added to this for fixed charges (interest and depreciation) on the station cost? Here again we can figure very closely.

Interest on the city bonds issued to build the plant may be taken at 4½%. A 5½% depreciation charge or sinking-fund charge (it makes no difference which it is called) will certainly be ample to replace the plant well before it becomes worn out or obsolete, as it would be sufficient to pay off the principal in about 14 years.

Assuming the total cost of the power plant including land and building at $50 per kw. of capacity (which is close to the actual figures) we have annual fixed charges

*A valuable and authoritative statement on the cost of power development with present economical machinery is given by Mr. H. E. Longwell, Consulting Engineer of the Westinghouse Machine Co., in a paper published in the "Journal of the American Society of Mechanical Engineers" for July. We quote the paragraph as follows:

"As regards the cost of power: If one is satisfied with investment charges of 10½% per annum, if he can buy really good coal at not to exceed $3 per ton, if he is a capable manager and a careful operator, and reasonably economical, he ought, with a plant of 6000- to 9000-kw. capacity size, to be able to produce a kw.-hr. at the switchboard with substantially 100% load factor, for around 4.3 mills."

The new Cleveland municipal plant will obtain its coal at a cost of less than $2 per ton, and its capacity is 25,000 kw. Both these factors will operate to reduce its operating expenses below the figure set by Mr. Longwell. On the other hand, of course, the plant does not operate with a load factor of 100%.

Since Mr. Longwell includes the fixed charges in his final figure of 4.3 mills it is evident that our estimate of 4 mills as the operating cost of the Cleveland plant with its advantage of cheap coal and large size units is too high.

per kw. of capacity = 10% of $50 = $5. Let us further assume a load factor of 33⅓%, giving an annual output per kw. of capacity of $\frac{365 \times 24}{3}$ = 2920 hr.. Then $5 divided by 2920 = about 1.6 mills.

Adding these fixed charges to the station-operating costs, we have a total cost of 5.6 mills for current at the station busbars.

And now what will it cost to distribute this current to the customers? Here we are on more difficult ground. The electric-lighting companies admit that the current which they sell the customer for 10c. costs them less than half a cent to produce in their stations; but they claim that the cost of distribution and sale is such that the 10-cent rate leaves them no more than a fair profit. But where are the actual figures of distribution cost to back up this assertion? Such figures appear to be jealously guarded.

Fortunately, Cleveland has experience enough of its own to serve as some guide in estimating these costs, for its Brooklyn municipal plant has now been in service for a dozen years.

As we have already noted, this Brooklyn plant, small in size and inefficient compared with present-day practice, nevertheless actually sold current to its customers last year at an average price of only 3.28c. per kw.-hr. and at this low rate earned a profit of 16% upon its book value.

Analyzing the operating expenses of the Brooklyn plant last year, we find that the total operating cost of both generation and distribution for the Brooklyn plant last year was 1.49c. per kw.-hr. and of this the station cost was 0.79c, leaving 0.52c. as the cost of distribution alone. This, however, was for current generated. The loss in distribution increases this to about 0.67 per kw.-hr. actually sold.

Now, it will certainly be agreed that a plant several times the size of the Broklyn plant can reduce the cost of distribution considerably below the figure quoted above. Let us be on the safe side, however, and assume that it will cost substantially the same or in round numbers 0.50c. per kw.-hr.

Next, what should be estimated for fixed charges on the distribution plant? The book value of the distribution plant in connection with the Brooklyn system is in the neighborhood of $100 per kw. of station capacity. Probably a considerably higher figure should be assumed for the cost of a new distribution system, as a large part of it will have to be underground. On the other hand, while the amount of capital required for an underground system is much greater than for overhead wires, the percentage of cost required to cover maintenance and depreciation is much less. To be on the safe side, however, we will assume for the distribution system to be installed for the new Cleveland municipal plant an average cost of $150 per kw. of capacity. Assuming the same interest and depreciation rate (10%) and load factor (33⅓%) that we assumed to find the fixed charges on the power station, we have $15 ÷ 2920 = 5.13 mills.

Summarizing now the several items, we have the following as the cost per kw.-hr. of output:

	Cents
Operating cost of generating current at station	0.40
Interest and depreciation on station plant	0.16
Operating cost of distribution	0.50
Interest and depreciation on distribution system	0.51
	1.57

This is the figure for current at the station busbars. If we assume a 20% loss in the distribution system, then the cost of current at the consumer's meter will be 1.96c.

Low though this figure may seem to be, we believe it is conservative and further that there is reason to believe the actual cost may be reduced below even this figure. The load factor, for example, may be 40% or more instead of the 33⅓% we have assumed, and this seems especially likely with the low rates made for large amounts of current for factory use.

It may seem on the face of the figures that the low rates of 1c. to 2c. per kw.-hr. for current to large users would result in a loss; but the cost to be figured against such consumers is chiefly the station-operating cost, since these customers use current at a time when the station and distribution system would otherwise be idle. The cost of distribution to such consumers also is generally very small.

Of course, only the test of final experience will determine what will be the average price received for the current sold, and, of course, this average price must be in excess of the 1.96c. found above for the average cost of current delivered to make the new municipal plant a financial success.

We may test the accuracy of these estimates also by comparison with the actual results in the Brooklyn plant. In this plant last year the entire operating cost of generating and distributing current was 2.06c. per kw.-hr. sold, and the allowance for interest and depreciation raised the total cost to 2.59c. If a 2500-kw. plant of out-of-date design can do this, it seems not unreasonable to expect that a 25,000-kw. plant operating with the high efficiency now obtainable will reduce this cost to 1.96c. The saving in station cost alone which will certainly be effected (the Brooklyn station cost last year was 0.97c. per kw.-hr. at the station busbars) would alone be nearly sufficient to reduce the cost as much as the estimate above given.

And if the Cleveland plant can deliver current to the consumer's meter at a total cost, including ample allowance for interest and depreciation, of 1.96c.; the prospects of the success of Cleveland's great experiment certainly seem favorable. The announced rate for the sale of current from the new station is 3c. per kw.-hr. Reductions from this are made only to consumers having connections of more than 50-kw. capacity. The flat 3c. rate will apply therefore to practically all private residences and all small stores and shops as well. The consumers who will benefit by the sliding scale and purchase current at less than a 3-c. rate are nearly all of them purchasers for power rather than for light.

It must be remembered, of course, that before the cost can be reduced to this figure, the station must secure a load substantially equal to its capacity and with sufficient power customers to raise the load factor to substantially the figure of 33⅓% assumed above.

The flat rate of 3c. per kw.-hr. will certainly be attractive to all those using current for residence lighting, but the users of current for power already have very low rates from the Cleveland Electric Co., and this part of the business will have to be be fought for. The crucial question for the Cleveland plant will be whether its reductions from its maximum rate to power users will so cut down

the average return that it will fall below the average cost of production and distribution. The managers of the Municipal plant have certainly showed shrewd business ability in so simplifying their rate schedule that the average consumer can understand it and know in advance the price he will have to pay. This feature alone will help to bring them customers.

It is said by the critics that Cleveland's municipal electric plant is a shrewd political move of Mayor Baker. To cut the cost of electric current to thousands of householders is an achievement which will surely add greatly to the popularity of the administration. We believe the managers of private companies selling electric current will do well to study the Cleveland situation and take a leaf from Mayor Baker's policy. The friendship and good will of the thousands of small consumers who purchase current is something which the private companies need to cultivate. At the present time the average householder feels that he is being made to pay three or four times as much for the current as is paid by the large manufacturers and department stores, simply because he is helpless, whereas the large user will put in his own electric plant unless the central station gives him a low price.

A cut in the price of current to the householder may serve to stop agitation for municipal ownership before it begins, and the price reduction will be to a large extent made good to the company in the stimulus given to a larger use of electric light and especially of electric heating appliances.

Doubtless the figures we have presented above will be criticized. The financial interests at stake are so large that this is to be expected. But in order that such criticism shall be valid, it should be not merely general but should show wherein and by how much the assumptions on which the above figures rest are in error. If facts and figures exist which would modify the above calculation, by all means let them be brought forward; and if 1.96c. is too low or perchance too high for the cost of current from Cleveland's new municipal plant, let it be demonstrated how the estimate can be made more accurate.

It may be said, for example, that a municipal plant will be free from the city, state and federal taxation which is laid upon a private company and that due allowance should be made for this in computing the operating cost of a municipal plant. Such a criticism is entirely valid. We have not included it in the above figures for lack of data as to the proper amount to allow.

It may be said also that the low figures of cost which we have computed above will not be attained until customers are obtained sufficient to give the station an economic load. While the low rates offered are sure to attract all the customers that can be carried, some time will be necessary, of course, to extend the distribution system and reach them.

Of course, the new municipal electric plant can only serve a small part of Cleveland's great area and 700,000 population. The Cleveland Electric Illuminating Co., with its 100,000-hp. station, and its distribution system covering nearly the entire city is doubtless in the field to stay. For a good while to come it will probably prefer to relinquish customers to its municipal competitor rather than lower all its rates to the level set by the latter. It has, however, already met the situation in part by a large reduction in its rate, which it is claimed to be equivalent to a reduction of $800,000 per year in the payments for electric current in Cleveland.

Admittedly, the situation of the companies operating electric stations when faced with such business-like municipal competition as Cleveland has undertaken is a difficult one. Besides the burden of taxation which we have referred to, the companies in most instances have put far more money into their plants than would be required to replace them with plants as good or better at the present day and their interest charges on capital are 2 to 3% higher than a city has to pay.

In the long run, however, replacement value and not the amount of money invested in the past must be the measure of safe capitalization. The rule which applies under free competition must be made to apply to public utilities also. If the price of the machinery equipment for an electric station is cut in two, then the capital ought to be reduced in proportion, even though the old steam engines and slow-speed generators and small-capacity boilers may have actually cost all they are capitalized for. If alternating-current distribution makes an old direct-current system obsolete, then its cost should be written off to profit and loss. Any concern which carries dead capital on its books at its original cost, or at any more than its true present value, is in a weak position whenever a strong new competitor appears.

It is this situation, in fact, which gives the greatest excuse for municipal ownership and operation of certain public utilities at the present time.

The general principle is unquestionably sound that all public utilities should be monopolies. Competition has been tried in water-supply, gas supply, street railways, telephones, and electric-lighting plants and has been proven in the long run to be a public injury instead of a public benefit. Economists and engineers are generally agreed that regulation of public utilities by public authorities rather than competition is the way out.

But it is not to be expected that a State public-utilities commission will compel a company to reduce its rates below the point necessary to yield a fair return on money actually invested; nor would the courts be likely to support such a forced reduction. If, however, a city like Cleveland can supply its citizens with electric current produced at lowest cost from an up to date plant there is no need of interference by commissions or courts.

And in conclusion, we should not by any means be understood as claiming that what Cleveland is doing in this field can be done by any other city anywhere. For one thing, Cleveland's advantages of cheap coal and abundant condensing water are great aids in reducing cost; but what is far more important is the success the city has attained in having its municipal enterprises carried out with a very fair degree of efficiency and honesty.

There will doubtless be other cities to follow Cleveland's lead in this experiment; but unless they are able to secure honest and efficient administration by competent engineers, and that not merely as a spasmodic effort at municipal house cleaning but as a permanent condition of city government—unless a city is able to fulfil this condition, experiments in municipal electric lighting will be apt to prove disastrous to the taxpayers.

Were it not for prevailing conditions in municipal government throughout the country, the electric-lighting companies would doubtless be even more concerned than they are over Cleveland's municipal electric plant.

Letters to the Editor

The SO_3 Content of Portland Cement

Sir—Prof. A. Baykoff, President of the Commission of the International Association for Testing Materials for the Study of the Influence of the Amount of SO_3 in Portland Cement, has communicated with the undersigned, who represent the American Society for Testing Materials on the Commission, in a letter which reads in part as follows:

As has been indicated in the report of the Association of German Portland Cement Manufacturers, various countries decide the question of the maximum amount of SO_3 in portland cement in a different way. Some make no difference between portland cement for fresh water and for sea water; others prescribe different limits according as the cement is to be used in fresh water or sea water.

Based upon these views, the VI Congress of the International Association for Testing Materias in New York adopted the following resolution:

> The Congress recommends generally that the conditions for accepting portland cement for all uses be fixed uniformly at a maximum amount of SO_3, that is to say, at 2.5%.

Thus the question proposed for the consideration of our Commission divides itself into two, which can be formulated as follows:

(1) Is it necessary to fix two different normals for the maximum admissible amount of SO_3 in portland cement according as the cement is designed for use in fresh water or in sea water?

(2) What should be the admissible maximum amount of SO_3 in portland cement under conditions conforming to the decision of the first question? Will it be necessary to use one or two normal figures?

As the solution of these questions can only be made after a sufficient quantity of exact data are available, it seems to me to be necessary, first of all, to collect a sufficient number of such data. With this object and independent of the researches which produce them or which are going to produce them, it will be useful to collect and discuss the greatest possible number of researches and the work devoted to the question of the influence of SO_3 on portland cement.

It seems to us that this matter should be put before the producers and users of portland cement in the United States so that such proposals in regard to the work to be done, and any data that may be available may be sent us for transmission to the President of the Commission. With this object in view, we are asking you to bring it to the attention of such persons through the columns of your journal. Communications are requested from all who are interested in the matter.

JOHN B. LOBER,
CLIFFORD RICHARDSON.

Woolworth Bldg., New York City, July 21, 1914.

Air-Valves and the Collapse of the Antelope Valley Siphon

Sir—In the valuable article on "Air Valves on Water-Supply Mains," by Clemens Herschel, in your issue of June 11, the writer notes the statement that the two ends of the Antelope Valley Siphon on the Los Angeles Aqueduct, which collapsed Feb. 18-20, 1914, were open. [See ENG. NEWS, Mar. 12, 1914, for account of collapse.—ED.] The fact is that this siphon did not have its ends open, but continued at each end into a closed gravity conduit, formed by excavation into the ground, having sides and bottom lined with cement concrete and the top covered with a reinforced-concrete slab.

It is therefore impossible to conclude that the Antelope Valley siphon collapsed from negative water pressure, and not from the effect of a partial vacuum.

The writer has frequently observed phenomena of negative presure in water pipes, as stated by Mr. Herschel, and, under certain conditions, where the factor of safety is low, as in this case of the Antelope Valley siphon, distortion of the pipe shell might occur from such negative pressure. It is extremely doubtful, however, if not entirely impossible, for one to account for such a complete collapse as that occurring on the Antelope Valley siphon without the formation of a decided vacuum inside of the pipe.

That the middle portion of the pipe remained in shape is not a satisfactory explanation in this case, because air would rush in through the lower end of the pipe, where the rapid efflux lowered the water, so it did not occupy the entire cross-section in the pipe.

F. C. FINKLE.

I. W. Hellman Bldg., Los Angeles, Calif., June 17, 1914.

Sir—The article in your issue of June 11, 1914, "Air Valves on Water-Supply Mains," by Clemens Herschel, in its reference to the Antelope Valley Siphon, Los Angeles Aqueduct, brings out a very interesting point: that of "negative pressure while the pipe was still full of water."

The ordinary gage used on pipe lines does not show the true internal pressure tending to distend the pipe, but a pressure which is lower by 14.7 lb., due to the fact that the gage is itself subject to an external pressure to that extent. The true zero internal pressure is 14.7 lb. below the zero of the gage, and in changing from a high pressure to the true zero the conditions, when passing the zero of the gage, will be the same as at any other point in the range of movement. Assuming an air-tight pipe, the action will be the same below the zero of the gage as above it. There will be no parting of the water column, or pipe walls and water, until the true zero is reached. It is assumed in this discussion that the water is free of air and other substances tending to change its condition.

The fact that the pipe becomes subject to a collapsing pressure at a certain point in this pressure drop is due to the counterbalancing of the pressures at this point, and has no other relation to the change within. It is therefore clear that, in the absence of air in the pipe, it is possible to have negative gage pressure to an extent of atmospheric pressure, 14.7 lb. per sq.in., without a parting of the water column, or a separation of the pipe walls and the water.

To what extent a pipe will permit a lower pressure within than without will depend upon its resistance to collapse. Should it be unable to withstand an excess ex-

ternal pressure of 14.7 lb., it will be necessary to prevent the internal pressure dropping to the true zero. The surest way to do this is to provide for the admission of air to partly counteract the external pressure, and *to do this wherever the internal pressure tends to go below the zero of the gage.*

It is, on first consideration, astonishing to find what a slight change in flow conditions is required to produce these low pressures. To illustrate, consider a pipe line on an incline of the same grade throughout, filled with water and having a nozzle at the lower end. Let the pipe be 5 ft. in diameter, ¼ in. thick and under a head of 200 ft. at the lower end. Let the nozzle be of such size that it will produce a flow of 5 ft. per sec. in the pipe.

Reference to the formulas in the article, "The Calculation of Water-Hammer Pressures," in ENGINEERING NEWS, Dec. 4, 1913, will show that in such a pipe a ram pressure of 34 lb. follows a check in flow of 1 ft. per sec. It is equally true that a sudden increase in flow is accompanied by a fall in pressure of 34 lb. This depends upon the same law which controls the ram, and is nothing more or less than the action of gravity upon a compressible fluid in an elastic pipe. These effects have been too thoroughly demonstrated by theory and experiment to be open to doubt.

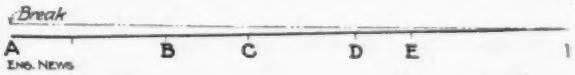
Suppose the nozzle to blow off, leaving a full pipe opening, and consider what will take place. Instantly the pressure of 87 lb. at the end of the pipe disappears, releasing the compressed water and permitting the pipe to contract, which two effects produce an additional flow of 2.54 ft. per sec. The pressure at the pipe end has fallen to atmospheric, which means that there is still an internal pressure of 14.7 lb. at that point. A wave of pressure drop, amounting to 87 lb., starts to pass up the pipe with a velocity of 2540 ft. per sec., but at a distance depending upon the inclination, a point is reached where 87 subtracted from the previously existing pressure reduces the internal pressure to the true zero. At this point, located about 34 ft. higher than the end, the water and pipe walls lose their power to furnish the required flow, and separation of the pipe walls and water takes place. It is seen that the pipe is subject to a superior external pressure varying from zero at the end to 14.7 lb. at the point mentioned.

Should the pipe be not sufficiently rigid to withstand this external pressure it will collapse and thus furnish the required flow with a less pressure drop. It is clear that the admission of air would produce the same result, in this respect, as the collapse, inasmuch as it would permit a partial evacuation of this part of the pipe to supply the required amount of water. Should no air be admitted and the pipe be strong enough to preserve its shape, a parting of the water from the pipe walls would result at the point where the true zero pressure was obtained, but not otherwise. The low-pressure wave will pass up the pipe, collapsing those portions of the line not sufficiently rigid to withstand it.

In this case an additional flow of 2.54 ft. per sec. (about 50%), brought about in the manner described, is sufficient to cause damage to a weak pipe line not provided with air valves in the proper place to care for such an emergency. This effect commences instantaneously and proceeds up the line with a velocity of 2540 ft. per sec. until the true internal pressure drops enough to permit the collapsing of the pipe by the external pressure or until the true zero is reached and separation takes place. In a uniform pipe this would probably take place between the upper wall and the water and not in the water itself. In lines consisting of pipes of varying size and thickness the conditions become very complicated, but, as a rule, it is possible to follow the effects until a collapsing pressure is found.

The point to be clearly understood is that the damage is done before any effect is felt at the upper end, and that prevention is to be found in the provision of adequate air valves along the line, even where the pipe will resist collapse. With the admission of sufficient air the pipe will flow partly full and the water will travel at a velocity dependent upon inclination and friction.

Referring to the Antelope Valley inverted siphon, the accompanying illustration shows the down-flow leg to the break and the table gives certain data pertaining thereto. Some of the figures are approximate only as they were scaled from a published diagram of small size. All pipe is 10 ft. in diameter.



DOWN-FLOW LEG OF ANTELOPE VALLEY SIPHON

Section	Length of pipe, ft.	Thickness of pipe, in.	Velocity of wave, ft. per sec.	Pressure change for 1 ft., lb.	Time of wave, sec.	Pressure at upper end, lb.
A B	2350	⅜	2300	31	1.02	78
B C	1415	⅛	2150	29	0.66	65
C D	1805	¼	1965	26.5	0.92	39
D E	891	¼	1965	26.5	0.45	33
E F	2736	?	2970	40	0.92	0

With such a break at the low point that free flow was permitted, the pressure at the end fell instantaneously to atmospheric, leaving an internal pressure of 14.7 lb., with no tendency to collapse. As the pressure fall was 87 lb. and 31 are required to give an additional flow of 1 ft. per sec., there was instantaneously added to the existing velocity a flow of 2.8 ft. per sec., calling for 220 cu.ft. per sec. This quantity would be provided by the pipe above the break to the limit of its capacity, which would be reached when the true internal pressure reached zero. As the pressure at *B* was 78 lb. per sq.in., there would have been, on the above assumption of free flow, a collapsing pressure of 9 lb. At *B*, where the wall thickness changes to $\frac{5}{16}$ in. and where a pressure fall of 29 lb. will increase the flow 1 ft. per sec., the required fall will be 81 lb., indicating a vacuum of 3 lb. In the ⅜-in. pipe the true internal pressure will be 5.7 lb. and in the $\frac{5}{16}$-in., 11.7 lb. There would be an equalization of these pressures, raising the former and lowering the latter.

According to a table recently published by Enger and Seely, the ⅜-in. pipe would partly collapse under such conditions, and the fact that it did not would indicate either that it was well supported, or, what is more likely, that there was not an entirely free flow permitted. To what extent the above conditions prevailed it is impossible to say, but it is evident that dangerously low pressures were promptly reached, and that had the lower sections been of sufficient strength to resist them, the usual lighter upper sections would have been damaged.

It is probable that the collapse started at the lower end and was completed during the rapid passage of the first wave of subnormal pressure; that air was supplied wher

the wave reached the top, and that thereafter the pipe ran partly full until it was emptied.

In this particular case, judging from the published accounts, there was absolutely no reason to anticipate such extraordinary flood conditions, and therefore no reason to provide for them. Economy limits the engineer to reasonable safety provisions, and loss from unusual manifestations of nature must be accepted as part of the penalty we pay for living. Such a risk should be insurable for the same reason that we provide against fire loss and life hazard.

As the upper ends of this pipe were open to the air and as there were no summits along the line, it is evident that the dependence which has been placed on air valves, located at the summits, is not sufficient protection against damage following extensive rupture at any point on the line. *The entire line must be provided, at intervals, with valves of sufficient size to admit enough air to prevent the collapsing pressure reaching the limit fixed by the rigidity of the pipe,* and this air must be admitted during the time required for the passage of the wave of subnormal pressure from the break to the top.

S. L. Berry.

317 Rialto Bldg., San Francisco, Calif.,
June 21, 1914.

[It seems to us that emphasis should be placed on a sentence in the last paragraph of the above letter which we have taken the liberty of italicizing. We do not believe that any computation can determine fully the particular points at which "negative pressure," which is really only another name for partial vacuum, may be developed under certain circumstances.

For example, a long level section of pipe may be flowing full, at high velocity under considerable pressure. If a break occurs on this section, the pressure will be at once relieved, and at some point beyond the break, the water will flow backward toward the break while the water beyond this point will continue flowing, impelled by the inertia of the whole moving column of water in the pipe below.—Ed.]

Making Cement Joints in Sewer Pipe

Sir—An interesting article, "Cement Joints for Sewer Pipe at Edmonton," appeared in your issue of July 9. The relative merits of certain details of procedure, under different specifications for pipe laying are discussed, and those which the author, J. M. Begg, has found the most satisfactory are incorporated in his quotation from the Edmonton specifications.

I have had some experience with cement-mortar joints made in three different ways: (1) where a gasket was not used; (2) where a gasket was inserted against the shoulder of the bell in the usual way; (3) where the gasket in the finished joint was at the entrance to the bell all the way round, calked flush with, or a little inside the outside edge of the bell. In all cases, the bottom third of the bell was lined with mortar before inserting the pipe. When no gasket was used, pipe 12 in. and over was centered by two small wooden chips. These chips were also used with the gasket in laying the larger sizes, 18 in. and over. Ordinarily, no difficulty was experienced in centering the 8-in. and 10-in. pipe without chips, whether a gasket was used or not. In every case the joints were pointed from the inside for all pipe 15 in. and over. In methods (1) and (2) the joints were protected with a burlap or cheesecloth band for pipe 18 in. and over.

So far as my experience goes, I am of the opinion (1) that the lower part of the bell should be filled before the spigot of the succeeding pipe is entered, and with more mortar than is needed in that part of the joint for safety the mortar should appear on the inside; (2) that it is important to use a gasket, not only to assist in centering the pipe, but I believe that the clogging, or silting up of a gasket calked tightly into the bell, is a considerable factor in preventing leakage in either direction through the joint, where the cement mortar itself is defective; (3) that while no doubt it can be dispensed with, it is safer to use chips to prevent the larger pipes settling at the spigot end; (4) that while it is proper to give the joint a finished appearance by a moderate bevel, the so called generous bevel is, if anything, worse than useless with the larger sizes, as it tends to sag away from the sides of the pipe; (5) that with the larger sizes, the joint should be supported by a strap of burlap, etc., unless a gasket is inserted at the face of the bell after the joint has been filled up the sides and over the crown.

Moreover, the third method of joint making has appeared to insure the tightest work so far as I have seen it used. This joint is made by lining the bottom third of the bell in the usual way, covering the gasket which is placed along the outside edge of the bell. The gasket is long enough to encircle the pipe, and the two end sections are brought out of the bell after passing clear of the mortar in the bottom. The spigot end of the pipe is then entered with its crown guided against the bell, worked down to center at both ends and bedded. After the joint has been completely filled with mortar, the free-end sections of the gasket are calked into the bell working from the bottom toward the crown. The gasket is driven flush with, or a little within, the face of the bell, the bevel applied, the joint pointed on the inside where accessible, and the surplus mortar cleaned out. There is not very much mortar driven through, particularly with the smaller pipe.

This feature of the mortar forcing through is occasionally objectionable in a way that is hard to avoid. The 12-in. pipe and under cannot be pointed from the inside, and if as sometimes happens, the plane of the spigot end is not perpendicular to the axis of the pipe, there may be a considerable space at the crown of the joint between the spigot end and the shoulder of the bell; and the mortar filling this space sometimes drops to the invert after the pipe laying has proceeded some distance in advance.

When laying 12-in. pipe, this feature has to be watched; it rarely happens with 8-in. pipe, or with the larger sizes where the joints have been pointed from the inside. With the joint made in this way the mortar is firmly held in the bell, and even if the bevel does sag, it cannot carry the mortar out of the bell with it.

I have not found the chips objectionable in the way Mr. Begg suggests; perhaps more mortar was placed in the bottom of the bell; at any rate the chips did not keep the pipe off the mortar; they were practically buried in it, near its upper edges. The chips were placed in this position, so that they could be easily moved a little higher, or lower, when occasionally required; moreover, with the

chips in this position, any inequalities in either the pipe or the chips do not raise or lower the spigot end so much as if the chips were placed lower down, opposite a diameter approaching the vertical. However, there is no doubt that with the chips omitted, the gasket will generally be fitted tighter in the bottom, and that is a desirable feature.

The consistency of the mortar is an important matter, and not at all easily controlled. The tendency is to have the mortar too soft, as it is easier for the pipe layer to handle. Unless quite stiff initially, the mortar which has been hurriedly mixed, becomes sloppy on further handling, and cannot be easily compacted or retained in the bell. The man who mixes the mortar often has too many other things to look after. The mortar should be mixed in very small batches, and kept thoroughly stirred some time before being used. This permits the mixture to absorb more water, and the proper consistency can be better gaged by the mortar man as further handling does not affect it; moreover, the mortar appears to be tougher, and works smoother.

Mr. Begg has adopted a good specification at Edmonton. Clause (e) which specifies that no joint is to be completed until at least two joints have been bedded and graded in advance is to be particularly recommended. In reading his well presented article, I had hoped to find the detailed description of the work extended to include the method or the procedure followed in bedding the pipe. It is in connection with this, that the most serious difficulties arise in laying sewer pipe. It requires considerable skill and patience to bed a pipe firmly, and at the same time have it lying true to grade and alignment. It is true, that once the pipe is in the right position, it can be held and supported by tamping proper filling around it with a thin-bladed tool. In this operation lies the greatest risk of lifting the spigot end slightly from the mortar in the bottom of the bell.

It has been the writer's practice to defer tamping immediately around the joint until after the mortar has set up; but in bright, hot weather, at least, the joints were covered with loose earth. Before back-filling the trench the following day, some of this filling was removed, before the tamping was completed. Possibly this was not a good system; care certainly had to be taken to maintain the compacted filling at the same level on both sides of the pipe, so as to avoid jarring it slightly off the line; it is surprising how easily this can be done with even a very light tamping iron.

W. S. Lea.

New Birks Bldg., Montreal, Can., July 19, 1914.

Sir—The editorial in the July 16 issue of Engineering News, relating to sewer-pipe joints, has been read with much interest by me as also the preceding week's article by Mr. Begg.

I am strongly in favor of incasing small-sized sewer mains and laterals in concrete as suggested by you, not only on account of its protection against seepage but also on account of safeguarding the pipes against almost certain breakage caused by workmen driving a pick though the pipe when a new ditch happens to cross the line of the old sewer. A covering of concrete is an immediate and positive warning of an existing sewer pipe and the workmen will naturally stop and find out what it is before digging it out.

My reason for answering this editorial is that I strongly object to the method suggested of covering the pipe with concrete as per the sketch on p. 147 of your issue of July 16, without at the same time calking the joint with jute and filling with cement which should be tied up tightly with a strip of muslin. The sewers under discussion would in general be from six to ten feet below ground and especially after a rain of any duration would be under from two to five feet of ground-water head and the covering of concrete could not possibly keep out the water. It is generally agreed that it is easy to keep water in concrete but a problem to keep it out.

It would seem, however, that the combination of all the methods suggested to date would make a good general working method of laying sewer pipes and insuring the maximum efficiency of the joints; namely, according to the size of the city, have one man definitely charged with the responsibility of having water-tight joints in his section, lay all pipes according to the very clear specifications suggested by Mr. Begg and finally to incase all mains and laterals up to perhaps 15 in. in concrete for protection from seepage and from tree roots.

Russell H. Peck, C. E.,
Engineer Palmerton Disposal Co.

Palmerton, Penn., July 19, 1914.

A Warning to Railway Construction Engineers in Pennsylvania

Sir—Referring to the letter of M. E. R. in the July 9, 1914, issue of Engineering News, relative to a warning to railway construction engineers working for English companies, which concludes with "It would be wise for men who have employment contracts under English law to insist on having their salaries paid at the end of the month," M. E. R. does not have to go to England for this kind of law. Certain employees of insolvent companies will find that the State of Pennsylvania has a similar law.

Some years ago, the undersigned was chief engineer of a Pennsylvania state railroad which failed and went into the hands of the sheriff for adjudication. Upon trying to collect his claim he found that the only preferred creditors were clerks, miners, mechanics and laborers; so although he had "labored" hard during his engagement, he had to be satisfied with a very small percentage of his claim. Under this law there is even a distinction between a clerk and a bookkeeper.

So long as engineering societies are nothing but literary clubs, no redress need to be looked for.

Civil Engineer.

Buffalo, N. Y., July 10, 1914.

Portland Cement Manufacture in the Philippines is one of the possibilities of the future. The "Philippine Journal of Science" for February contains an article by Augustus T. West and Alvin J. Cox describing tests of raw materials available in the Philippines for portland-cement manufacture which showed excellent results. All the cement now used in the Philippines is imported. The consumption in the fiscal year 1913 was just short of 400,000 bbl. Cement from the United States is imported into the Philippines free of duty but the transportation cost is so great that American cements do not compete in the Philippine market. The selling price of cement in the Philippines on large contracts is stated to have varied from $2.18 in 1907 to $3.34 in 1912.

Sewer Explosions and their Prevention*

By Leonard Metcalf and Harrison P. Eddy†

SYNOPSIS—This article, composed of extracts from a forthcoming book on "Modern Sewerage Practice," gives in detail the precautions to be observed by sewer workers to avoid the danger of explosions. The nature of sewer explosions is gone into and the Washington City ordinances for preventing explosive mixtures in sewers are quoted. Dangers of asphyxiation are also described and precautionary measures outlined.

When a sewer is to be entered, or a lighted lamp or candle is to be lowered into it, great care should be exercised to avoid danger of explosion. Men accustomed to this work will quickly perceive the presence of illuminating gas or gasoline vapor, by the odor. This is the safest method of testing the air, but the observer should be careful to note whether the air is entering or emerging from the manhole, otherwise he is likely to mistake the absence of odor for the absence of gas in the sewer. If it is necessary to enter the manhole to determine the presence of gas, this should be done without a light, unless it be an electric lamp, and the man entering should wear a life belt with a line leading to his helpers at the top of the manhole. Although seldom done, it is always wise to open several manholes on the sewer to be entered some little time before the entrance is to be made, in order to provide a change of air within the sewer.

An inspection in, or very close to a manhole, should never be attempted by one man alone; the inspector should have at least one helper both for his protection and that of the public, and usually more than one. The helper should remain standing at the top of the manhole to warn approaching persons of the opening. Where the inspector is to go from manhole to manhole through the sewer a second helper should precede him to the manhole below that at which he enters. When the inspector has reached the second manhole, the second helper signals the first one at the first manhole, who replaces the lid and passes on to the third manhole, awaiting the arrival of the inspector at this point. Three men constitute the minimum safe size of gang for such inspection, and preferably two men should go together through the sewer.

Danger of Explosions

Many explosions have occurred in sewers, due to explosive mixtures of illuminating gas, naphtha vapor and possibly other gases. Illuminating gas leaking from a defective gas main may find its way through the ground and into a sewer in sufficient quantity to cause an explosive mixture. Most sewer-maintenance superintendents have had experiences of this kind, although relatively few instances have resulted in explosions. It is not always easy to find the source of such gas, as it may travel a considerable distance from the point where it enters the sewer to the place where it is detected.

During recent years, trouble has frequently resulted from the discharge of gasoline into sewers, largely from automobile garages. Gasoline vapor is considerably heavier than air and therefore is not always readily distinguished by its odor when the observation is made from the top of the manhole. This new source of danger has necessitated ordinances preventing the discharge of gasoline into sewers and approving lighting appliances for the use of inspectors and laborers while working within the sewers.

An explosive mixture may result from the decomposition of the organic matter in the deposits. By such decomposition, marsh gas, or methane, may be produced in considerable quantities, and if there is not an adequate circulation of air to provide ventilation, it is conceivable that the proportion of this gas to the oxygen in the air in the sewer may be sufficient to provide an explosive mixture.

On Nov. 25, 1913, what was probably the most disastrous sewer explosion on record up to that time occurred in Pittsburgh, Penn. A detailed account of this accident was printed in Engineering News, Jan. 1, 1914. A series of spectacular explosions occurred in sewers in New York City, on Oct. 7. 1909, which was described by A. A. Breneman in Engineering News, Dec. 2, 1909, p. 608. The conclusion seemed to be that these explosions were due to gasoline vapor.

H. J. Kellogg, Assistant City Engineer, New Haven, Conn., carried on investigations for about a year to determine causes of, and to find means for preventing, explosions in sewers. He reported some of the results of these investigations in a paper read before the Connecticut Society of Civil Engineers, Feb. 10, 1914, from which the following notes and comments are taken:

> In 1886 there was an explosion in a New Haven sewer 60 in. in diameter. The arch was thrown completely off for about 100 ft. No one knows what combustible wrought the destruction, nor how it was ignited. No one was injured. . . . About 1894 a 54-in. circular brick sewer was being cleaned by four men when an explosion occurred. The men escaped with difficulty, two of them being disabled for six months or more. The cause of the explosion appears to have beeen illuminating gas ignited by lanterns which the men carried. . . . In 1913 a 42-in. egg-shaped New Haven brick sewer was being cleaned when an explosion occurred, burning the men about their hands and faces. The cause of this explosion does not appear to have been definitely ascertained, although a slight odor of gasoline was noticed before the men entered the sewer. If gasoline was the cause of the explosion, it probably came from a manufacturing plant, the drain from which entered the sewer close to the point at which the explosion occurred. Very small quantities of gasoline are used at this plant.

At Philadelphia, an explosion in which several men were injured was attributed to gasoline vapor. Mr. Kellogg quoted from a letter from George S. Webster, Chief Engineer, Bureau of Sewers, as follows:

> A suspicious smell was noticed upon opening a sewer manhole, and a blazing paper thrown in resulted in a column of flame about 40 ft. in height, due to gasoline from a nearby factory.

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* Reprinted by the kind permission of the authors and publishers from a forthcoming book on "Modern Sewerage Practice," by Leonard Metcalf and Harrison P. Eddy. McGraw-Hill Book Co., publishers. The extracts are from Chapter 19, "Operation and Maintenance of Sewerage Systems."

† Consulting engineers, Boston, Mass.

Two men were killed in a Philadelphia sewer explosion about 1912. The cause is said to have been illuminating gas from a leaky main which ran parallel to the sewer. At a later date a man was burned by an explosion resulting from gasoline discharged from a dry-cleaning shop.

Explosions in sewers were reported from Providence, R. I., Buffalo, N. Y., Washington, D. C., Fall River, Mass., Detroit, Mich., Brooklyn, N. Y., Cleveland Ohio, Worcester, Mass., St. Paul, Minn., Kansas City, Mo., Duluth, Minn., Los Angeles, Calif., Savannah, Ga., Baltimore, Md., Hartford, Conn., New Haven, Conn., Pittsburgh, Penn., San Francisco, Calif., Louisville, Ky., and Charlestown, S. C. The cause of explosion in a majority of these places was reported as illuminating gas.

George H. Norton, Deputy Engineer Commissioner, Buffalo, N. Y., described a new gas found in sewers as follows:

It may be interesting to know that for three years past, at periods following thawing weather, after frost and heavy snow, we have had serious complaint from gas odor in some of our sewers. This has been traced and found to be due to the use by the railroads of what is called a hydro-carbon oil, which is used by them to prevent freezing of their switches and interlocking plants in their passenger station yards, and at other congested points. An examination was made by the city chemist, and he reported that at low temperatures this oil will crystallize, that such crystals are soluble in water, and for this reason he thinks that the odor or gases have passed the ordinary vent sewer trap. The gases arising from this oil are apparently highly explosive and would readily be the cause of a serious explosion. This oil is a byproduct from the Pintsch gas used for car lighting.

Roscoe M. Clark, City Engineer of Hartford, Conn., gave the following information:

I have no recollection of any explosions in the sewers previous to 1913. In 1913 we had two serious explosions in the Park River intercepter. The first, occurring in January, lifted one manhole head and threw several covers in the air, and also shattered windows in adjoining houses. The second occurred in May, when the East Side Pumping Station and its connection with the Park River intercepting sewer were in the process of construction. The most damage was done to the station and adjoining houses. Both of the explosions took place in the vicinity of the works of the Hartford City Gas Light Co. Suits brought by property owners and the contractor for the pumping station are now pending in court.

Robert Adamson, Fire Commissioner of the Borough of Manhattan, New York City, reported that there were eleven sewer explosions between June 5, 1912, and the date of Mr. Kellogg's inquiry, probably about a year later. He stated that the probable causes of explosions in New York are gasoline, illuminating gas, and calcium carbide. He stated further that prior to the regulation requiring the installation of oil separators in garages, the majority of sewer explosions was limited to what is known as the "garage zone." Since their installation, the number of explosions has been greatly reduced. The following quotation is from Mr. Adamson's letter:

The illuminating gas enters the sewer from ruptured, corroded or broken gas mains, or from leaky joints, and when mixed in the proper proportions with air forms an explosive mixture. Calcium carbide, which is used in garages for generating lights, may be thrown into the sewer, and when in contact with water generates acetylene gas, which is an intense explosive and is further auto-combustible, so that in many instances it may be lighted by its own heat generated in evolving the acetylene.

The vapors of gasoline and illuminating gas require an open flame to ignite them, but whether this occurs in sewers it is impossible to tell. In vaults and in conduits the igniting spark is furnished in a number of cases by electrical apparatus, such as switchboards, sump pumps, short-circuiting, etc.

Some relation exists between sewer explosions and flooding of the sewers, and further, when the sewers become tide-bound these explosions are more frequent, such as following an unusual shower of heavy rain or a fall of snow or thawing of ice. This causes a condensation of the gases within the sewer.

There are no causualties on record resulting from these explosions, but considerable damage to property often occurs.

In relation to places where oil is stored, the oil separator is the only method at present under the regulations of preventing volatile inflammable oil from discharging into the sewer.

Mr. Kellogg stated:

That one volume of gasoline oil produces 141 equal volumes of gas or vapor, as figured by the city's chemist, who gave as the computed best mixture for complete combustion: 1 part of gas to 62½ parts of air. In gas engines, especially in automobile driving, an excess of air is said to give better practical results. The following figures lead up to a knowledge of the amount of gas likely to be in a sewer from a given amount of oil: 1 gal. gasoline gives 141 gal. of gas; 141 gal. gas equals about 18.8 cu.ft. of gas; 18.8 cu.ft. of gas with 1175 cu.ft. of air gives the best explosive mixture.

There is a broad variation in the proportions of this gas and air that will form combustible mixtures. Now, allowing 8 cu.ft. as the available space for 1 lin.ft. of a 42-in. sewer, in New Haven, 1 gal. of gasoline could have furnished a prime mixture for about 150 lin. ft. of that sewer. A leaner but still dangerous mixture might more than double the danger zone. Although the rate of flow in the sewer, the time it would take to vaporize the oil, the draft in the sewer and perhaps other factors complicate the problem, the above figures show that a comparatively small quantity of gasoline is capable of making trouble.

Mr. Kellogg also stated that about one part of illuminating gas to about seven parts of air is computed to be the best mixture for complete explosive combustion.

The following ordinance from Washington, D. C., appears to provide as ample protection as is possible by law:

Sec. 18. No persons shall make or maintain any connection with any public sewer or appurtenance thereof whereby there may be conveyed into the same any hot, suffocating, corrosive, inflammable or explosive liquid, gas, vapor, substance or material of any kind; and no persons shall cause to enter or flow into any public sewer or appurtenance thereof any hot, corrosive, suffocating, inflammable or explosive liquid, gas, vapor, substance or material of any kind; provided, that the provision of this paragraph shall not apply to water from ordinary hot-water boilers of residences.

On Jan. 3, 1912, the Municipal Explosives Commission of New York adopted a code of regulations, some of which bear directly upon the admission of inflammable oils to the public sewers.*

The "Paragon" gasoline and oil separator is an apparatus which has been put upon the market by the Ansonia Manufacturing Co., of New York, to meet the regulations of the New York City ordinances. In it, by taking advantage of the difference in specific gravity, the oil and gasoline are allowed to overflow into an oil chamber while the water passes through the trap and out into the sewer. From time to time, as oil and gasoline accumulate in the chamber, they are drawn off or pumped out.

The occasional presence of illuminating gas in sewers has been a matter of common knowledge for a great many years. It does not seem that there is at present any tendency toward an increase of this sort of trouble, and so far as the authors know, there is no means of preventing its occasional occurrence. It is usually discovered and reported before an explosion occurs. When reported, every effort should be made to immediately ventilate the sewer in which it is found and to locate the leaks in the main which is permitting the gas to escape.

* Printed in "Engineering News," July 2, 1914, p. 13.

The trouble arising from gasoline vapor in sewers appears to have increased greatly in the last few years with the increased number of gasoline engines and automobiles in use. Something toward decreasing the danger can be accomplished by means of oil-separating traps thoroughly well ventilated to prevent the escape of gasoline into sewers. This does not seem to have met with general application, however, and great care should be exercised by those entering sewers to avoid danger of explosions. One of the best means of accomplishing this is to prevent open flames in sewers by using the electric lantern.

An instructive discussion by D. B. Rushmore, of safety lamps and a new electric light for miners, appears in ENGINEERING NEWS, Aug. 15, 1912. The electric lamp, made by the General Electric Co., is a combination of the hand lantern and headlight, consisting of a miniature tungsten-lamp unit operated from a light storage-battery. The battery has a capacity of 5 amp.-hr., and is of sufficient size to operate a lamp from 12 to 14 hr.

When used as a headlight, the lamp is fastened to the miner's cap and connected by a flexible chord to the battery, which is flat, 6½x4⅝x1⅛ in., weighing 3 lb. and is carried on a belt. When used as a hand lantern, the lamp socket is removed from the cap receptacle and inserted into the receptacle on the side of the battery, simply taking the place of the cable attaching plug.

OTHER SEWER DANGERS

Even though the danger of explosion is avoided, in exceptional cases gases may be present in sufficient quantities and of the proper kind to endanger the lives of the men working in the sewer. Illuminating gas, while most common, may usually be detected by its odor. If, however, there are some gas-works wastes in the sewage which impart odor to the air and yet are not dangerous, the men may gradually come upon a quantity of illuminating gas without noticing the difference in odor, and be overcome.

There is also danger from marsh gas formed in decomposing deposits and liberated in large quantities when these deposits are disturbed, as by being shoveled or walked upon. Hydrogen sulphide may also be formed under similar conditions, especially where the sewage contains sulphates, as from very hard water or sea water. An illustration of such conditions is furnished at the end of the Los Angeles, Calif., outfall.

Another illustration is afforded by the death of three experienced sewer workmen at New Bedford, Mass., on Apr. 10, 1914. A section of an intercepting sewer had been closed by a bulkhead in order that the portion below the bulkhead might be put in use while that above was being completed. The men entered the sewer at a manhole 500 ft. above the bulkhead. They adopted none of the precautions advised in this chapter. About 8 hr. later, their bodies were found by a searching party under conditions which indicated that they had probably worked their way well down toward the bulkhead, when they discovered the presence of gas. One of them escaped to within 100 ft. of the manhole and then fell over with his face in a little trickle of water on the invert, in which he was probably drowned. The others did not get nearer than 300 ft. to the manhole; their bodies were found against the wall of the sewer, the arm of one through that of the other in a way indicating that he was helping the latter along when both collapsed.

The only practicable way in which to avoid danger of being overcome by such gases as may be found in sewers under exceptional circumstances appears to be to provide liberal ventilation before entering and while working in them. Care should also be exercised to provide young and vigorous men for such work, to see that there are sufficient men in the party to provide necessary assistance in case of need, and to require that life belts be used, when the men are in particular danger. For emergencies, the breathing apparatus used in mine-rescue work and for entering burning buildings may prove useful. Such apparatus is described in Miners' Circular 4, U. S. Bureau of Mines.

Effect of High-Pressure Steam Mains under Streets

Damage to streets, sewers and paving by the presence of high-pressure steam mains in the street having been proved by an extensive investigation, the Public Service Commission for the First District, New York, on July 23, ordered the New York Steam Co. to remove and replace its 58,986 ft. of steam mains of the brick-trench type with mains of the modern tile construction shown on the accompanying drawing, or by some other type equally good, to be approved by the Commission, within five years, at the rate of not less than 12,000 ft. per year, beginning Sept. 1, 1914.

The New York Steam Co. was formed back in 1880. It is said to be the only concern of its kind operating high-pressure steam lines for heating and power. Three steam-boiler plants are operated. One, known as station B, built about 1885, in the lower part of the city, has a rated boiler capacity of 15,800 hp. The second, station J, built in 1897, with a rated capacity of 11,700 hp., is at the foot of 59th., East River. The third is at about the same location and is called station J Addition. It was built in 1908 and has a rated capacity of 4800 hp.

Station B supplies steam at 95 lb. boiler pressure with 70-lb. pressure at the extreme end of the mains. The steam from this plant is used about equally for heat and power. The other two plants serve the uptown district with steam at 55 lb. boiler pressure with 40-lb. pressure at the end of the mains. The steam is used almost exclusively for heating. The temperature of the steam in the mains is over 300° F. In addition to these stations, the company owns three ice plants with a combined daily capacity of 195 tons. The total value of the property of the company is estimated at $2,168,000.

The chief engineer of the company stated that the cost of relaying a 10-in. (the average size) main was about $18 per foot, and that this cost was greater than that of laying new mains because of the necessity of maintaining service and of removing the old pipe. In the uptown district there were 625 customers, 172 of which were supplied through meters; the others were served at flat rates. In the downtown district, there were 456 customers, all supplied through steam meters. It was stated that the maximum price charged for steam is $2 per thousand pounds. The minimum rate is 47c.

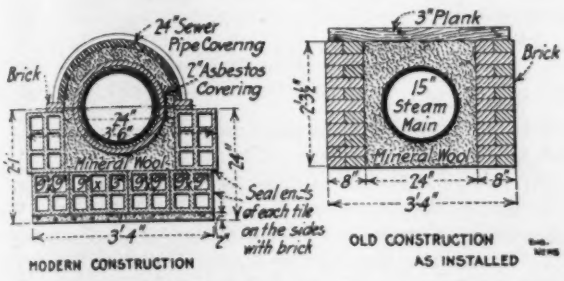
CONDITION AND EFFECT OF STEAM MAINS

The investigation revealed that it was a common occurrence to find the earth, for some feet outside the brick

walls of the steam mains, filled with steam which escaped when the earth was disturbed, and that the temperature of this earth ranged from 85 to 193° F. even in winter. One effect of these mains was to burn the leather washers in the water hydrants, and water discharged from some high-pressure hydrants was hot enough to give off steam. The unsatisfactory condition of the wood-block pavement in certain streets and their excessive cost of maintenance, can be attributed largely to the effect of the steam mains.

At street intersections, the waterproofing of the subway on lower Broadway was damaged by heat from the mains; the asphalt had practically disappeared, and the fabric was dried up and cracked. This effect is most important, because much of the new subway now under construction on lower Broadway is below high-water level, and waterproofing will, therefore, be necessary. This new line will encounter many steam mains between Chambers St. and Park Place, and elsewhere.

As for the effect of the steam mains on pavements, it was observed that on damp days such pavements were



PRESENT MAIN CONSTRUCTION OF N. Y. STEAM CO. AND NEW TYPE WHICH WILL REPLACE IT

dry, and the manholes in the street emitted steam, and that a light fall of snow would melt quickly.

In a certain sewer manhole back of Trinity Church, the temperature was 132° F., and in the sewer nearby was 180°. This temperature prevented men from working in the manholes. Another effect of this high temperature in sewer manholes was bad odors which issued from them. It was stated to be the consensus of opinion in the Department of Public Works, New York City, that the steam will loosen the cement in the joints of the sewer brickwall. In another manhole a 3-in. drip-pipe was discovered protruding through the wall and discharging into the sewer, steam at 210° F.

In the manholes of electric-cable subways, the temperature from the steam mains reached as high as 180°. The effect of this temperature is not only to prohibit men from working in the manholes, but also to prevent the cables from working to full capacity.

It was stated that 60% of the steam pipes had screw flanges, which frequently leaked at the threaded connections. The chief engineer of the company said that welded flanges came in use on the system in 1901; and that about half of the mains in both districts had welded flanges and were laid in drained trenches. The number of feet of mains of modern tile construction in the downtown district is 3796, and in the uptown district 10,751, a total of 14,547 out of 73,553 ft., or less than 20%. The chief engineer of the company claimed that no leakage troubles were experienced where the trenches were drained and welded flanges used. His explanation of certain steam leaks which were proved to exist was that the trenches were not drained there, and that, although the pipes had welded flanges, they were the first of that kind so fitted, and that the flanges were made of wrought iron and had "porosity in them." The steam company claimed that much of the damage caused by leaks and radiation was because other subsurface structures have been located too close to the steam mains.

The report goes on to say that:

> subsurface structures in many streets of New York are numerous, and space is limited. Each must be so constructed as not to injure the others, and that the record shows that no injury would have been caused had the steam mains been properly constructed. The steam company also says that much of the trouble is caused by leaks from water mains and house service piping, which is notoriously bad. But water pipes cannot be dispensed with, and the business of the steam company must adjust itself to conditions as they exist.
>
> Mr. Goodrich [Consulting Engineer to the President of the Borough of Manhattan] testified that at Detroit, Mich., steam mains are placed in a tunnel which is from 15 to 35 ft. below the surface of the street, and that it would be feasible to construct such tunnels in New York.

The recent inquiry has been limited to the question of improvement in the distribution system of the steam company. No investigation has yet been made of the generating plants, or of other methods employed by the company in the conduct of its business.

New Locomotives for the Pennsylvania R.R. have been designed specially for use on the main line between Pittsburgh and Altoona, the purpose of the freight engines being to reduce double-heading to a minimum and to avoid breaking up trains at these points owing to the severe grades on this division. The clearance limits are restricted, and the weight per driving axle is limited to 65,000 lb. (with a 5% margin for scale variation), and the dynamic effect of the unbalanced reciprocating parts at 70 m.p.h. is limited to 30% of the weight on drivers. It was necessary to increase the power without exceeding the present clearances and to make the moving parts as light as possible while maintaining ample strength.

The running gear has been lightened by the use of heat-treated steel for driving axles, crankpins, piston rods, and connecting side rods, while the axle, pins and piston rods are hollow. The engines have the Walschaerts valve-gear, Schmidt superheaters, firebrick arches and screw reversing gear. The cabs are of steel, and much shorter than the ordinary design, this being made practicable by the use of the power reversing gear. The boilers and tenders are the same for both classes, and the boilers have the Belpaire firebox, 80x126 in.

The freight engines are of the 2:8:2 class, superseding a 2:8:0 class having cylinders 25x28 in. and a driving-wheel load of 220,000 lb., as against 27x30 in. and 262,000 lb. in the new engines. The tractive power is 57,850 lb., and 46,290 lb. for the new and old engines respectively. The passenger engines are of the 4:6:2 class, taking the place of a 4.4:2 class having cylinders 23½x26 in. and a driving wheel load of 133,100 lb., as against 27x28 in. and 200,000 lb. for the new engines. The tractive power is 41,845 lb. for the new and 29,427 lb. for the old engines. The leading dimensions of the new engines are as follows:

	Freight	Passenger
Class	2:8:2	4:6:0
Weight, total	330,000 lb.	305,000 lb.
Weight on driver	262 000 lb.	200,000 lb.
Wheelbase, driving	17 ft. ½ in.	13 ft. 10 in.
Wheelbase, engine	36 ft. 4½ in.	36 ft. 2 in.
Wheelbase, engine and tender	72 ft. 3 in.	71 ft. 10 in.
Cylinders (2)	27x30 in.	27x28 in.
Driving wheels	5 ft. 2 in.	6 ft. 8 in.
Boiler diameter	6 ft. 6½ in.	6 ft. 6½ in.
Working pressure	205 lb.	205 lb.
Tubes, length	19 ft.	19 ft.
Heating surface, tubes	3747 sq.ft.	3747 sq.ft.
Heating surface, total	4036 sq.ft.	4036 sq.ft.
Superheating surface	1154 sq.ft.	1154 sq ft.
Grate area	70 sq.ft.	70 sq.ft.
Rail to center of boiler	9 ft. 9 in.	10 ft. 1 in.
Water in tender	7000 gal.	7000 gal.
Coal in tender	25,000 lb.	25,000 lb.

The Ohio River Bridge at Metropolis; Record-Breaking Simple-Truss Span

One of the most important of the many large bridges now projected and under construction is the new Ohio River bridge at Metropolis, Ill., the work for which has just been commenced and which will form part of a new railway route from northern and central points to the Gulf. The bridge will be a double-track structure, designed for unusually heavy loading and having a truss channel span 722 ft. 11 in. long between centers of piers. This span, being some 50 ft. longer than the main truss spans of the Municipal Bridge at St. Louis, will be by a large margin the longest simple span ever built.

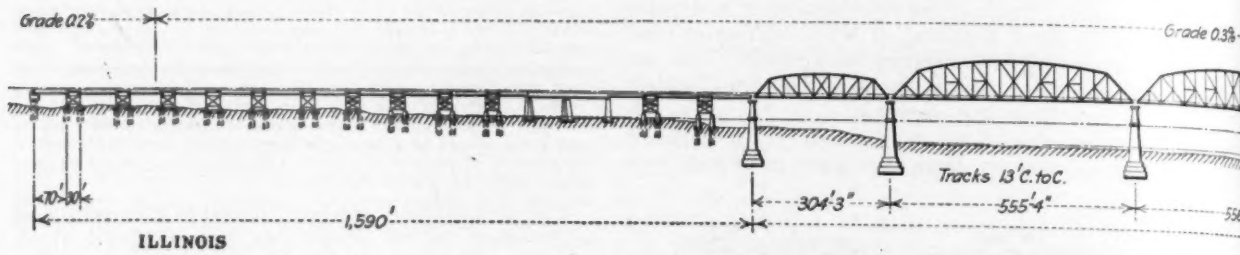
The Metropolis Bridge will be built and owned jointly by the Chicago, Burlington & Quincy R.R. and the Nashville, Chattanooga & St. Louis R.R., and will form part of a new line between Metropolis, Ill., and Paducah, Ky., a distance of 15 miles. It is the development of a project initiated about 2½ years ago to establish a new and direct railway route between Chicago and Gulf ports (ENGINEERING NEWS, March 21 and May 2, 1912). This project was for a double-track bridge near Paducah to be built jointly by four roads (the Chicago, Burlington & Quincy, the Chicago & Eastern Illinois, the Illinois Central and the Louisville & Nashville). The bridge with its railway connections was to form the main part of a link between lines north of the Ohio and the New Orleans, Mobile & Chicago Ry., a new line owned by the 'Frisco and Louisville & Nashville railways. Financial and other troubles broke up the original project, but it is now to be carried out on practically identical lines.

The bridge proper will have a channel span of 723 ft., four truss spans of about 550 ft., with a 300-ft. shore span at the north end and a 250-ft. shore span at the south. This last is a deck span, while all the others are through spans with pin-connected trusses of the Petit type. The trusses of the through spans are spaced 37, 33 and 32 ft. c. to c. for the 720-ft., 551-ft. and 300-ft. spans respectively, while the maximum truss depths are respectively 110 ft., 83 ft. and 48 ft.

The north approach is a steel viaduct about 1600 ft. long, having 30-ft. tower spans and 70- to 90-ft. intermediate spans; at one point (where the line crosses a railway) the spans are carried by masonry piers instead of the steel towers. The south approach is similar, with alternating 30-ft. and 75-ft. spans, but having a length of only 600 ft. These approaches will have a concrete deck for ballasted tracks. The total length of bridge and approaches is about 5700 ft. The bridge is practically level, with ascending approach grades of 0.2% at the north and 0.3% at the south. The clear headway above high-water level is 53 ft., or 112 ft. at low water.

The foundations for the seven river piers will be sunk through a firm sand and gravel formation, pneumatic caissons being used; the maximum depth below low water level is about 78 ft. The pedestals of the viaduct approaches will be supported on pile foundations, which will be built by the railway company.

A special feature of the bridge is the very heavy live-load assumed in the design, this being (for each track) two engines of Cooper's E-90 loading followed by a train load of 7500 lb. per lin.ft. This was adopted in view of the increasing weight of locomotives and trains,



THE OHIO RIVER BRIDGE AT METROPOLIS,
(C. H. Cartlidge, Chief Engineer; Ralph

and to maintain uniform unit-stresses under these future heavy loads.

The bridge will be built by the Paducah & Illinois R.R. Co., a subsidiary of the Chicago, Burlington & Quincy R.R., and the Nashville, Chattanooga & St. Louis R.R. The Chief Engineer is C. H. Cartlidge (Bridge Engineer of the C., B. & Q. R.R.), and Ralph Modjeski is Consulting Engineer. The total cost will be about $3,500,000 for the bridge and approaches. The contract for the seven river piers was awarded in June to the Union Bridge & Construction Co., of Kansas City, Mo. Bids for the superstructure (the seven truss spans only) were received July 20, and the contract probably will be awarded in a few weeks. The bridge is to be completed in 1916.

Graphite Treatment for Boiler Scale

BY L. W. BROOKS*

Much has been heard in the last two years of the use of graphite in removing scale and sediment from steam boilers, and the writer has felt that this treatment should be of great interest to readers of ENGINEERING NEWS, comprising as they do a large number of water-works engineers and general contractors.

Many believe the scheme to be a new one but really it dates back 25 years. Active interest in preventing boiler scale continues for two reasons: (1) reduction of efficiency—a deposit of scale $\frac{1}{16}$ in. thick increases the fuel burned by 5 to 15%; (2) decrease in safety of operation—for scale is so poor a conductor that the boiler metal may be overheated.

In most plants the foremen have their own favorite methods of attacking the scale problem. Some are partly

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successful and others are useless if not actually harmful. In any case their success or failure is largely a matter of chance. A hit-or-miss attack is fairly certain to miss. Of the ordinary preventives, preheating the feed water, filtration, use of kerosene, introduction of zinc into the boilers, and chemical treatment of the water either inside or outside the boiler are the most common. Most such remedies are only a partial success for they do not remove all the scale, while some of the inside chemical treatments really do more harm than good.

The writer would not intimate that a chemical compound cannot be satisfactory. For instance if a water, known to contain a reasonably constant proportion of sulphate of lime, were treated with the proper amount of carbonate of soda, no hard scale would be produced. Grains of carbonate of lime would fall down but would adhere to the plates and could, therefore, be blown out or gathered into sediment collectors. Chemicals would be in better repute if they were used more intelligently but unfortunately they are seldom compounded with reference to all the local conditions. The damage done to boilers by unsuitable compounds is enormous.

In contrast to such doubtful remedies the graphite treatment is conspicuous. The principle upon which it works is rather unique. It does not prevent the formation and precipitation of solid matter but it coats the interior of the boiler below the water line with a slippery veneer to which the scale cannot adhere firmly. Particles of graphite also settle on the bits of scale as they form and make the deposit soft or crumbly. As a result boilers can be cleaned with remarkable ease. Men who have cleaned boilers with hammer and chisel, or turbine cleaner, appreciate the difference. The writer has in mind one case where formerly five men required three days to clean the 580 caps of certain water-tube boilers. Since using graphite three men clean them in one day. Instances of this kind are common.

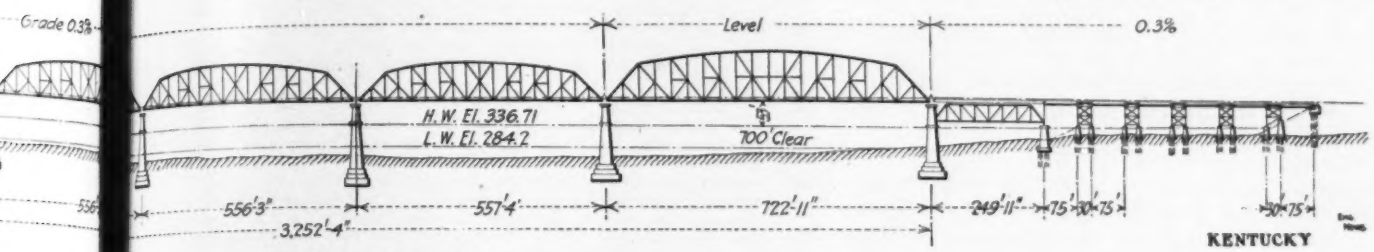
Graphite has no chemical action whatever. One would hardly believe it possible that graphite can remove old scale that the so called compounds have failed to loosen. The theory of its action is interesting. From the inside of the boiler, scale is seen to be full of fine checks or cracks, appearing somewhat like alligator skin. Contractions and expansions of the shell and tubes loosen the small sections at the edges. Graphite being swirled around in the water finds its way into the cracks and gradually gets under the edges of the sections of scale, thereby preventing the edges from sticking to the metal again. Continual expansion and contraction, with graphite filling the cracks, loosen the sections still more; graphite creeps in farther until eventually whole sections of scale are separated from the metal and they fall to the bottom of the shell (in return tubular boilers) or may be easily rapped loose. The writer knows where scale has been thus removed from water-tube boilers in which it had accumulated so thick that there was only a small hole left through the center of the tubes. To be sure, it took considerable time. The particular plant in which this was done is one of the thousands in which compounds have been used for years but have failed to prevent scale.

Graphite will not cause pitting or corrosion, is an excellent conductor of heat, prolongs the life of gaskets, packings and valve seats at least ten times their usual life, and in no way affects the quality of the steam. This latter virtue is welcome in plants that require clean steam —such as canneries, laundries, ice plants, and sugar refineries.

In beginning a graphite treatment it should not be expected to bring down the scale overnight or even in a week. It is reasonable to wait from three to ten weeks, according to the thickness and hardness of the scale, before appreciable results may be expected. Boilers operated continuously yield to the treatment more slowly than those in which the fires are banked overnight because of great dependence upon repeated expansion and contraction of the metal to crack the scale. Ordinarily the new scale deposits as sludge or in small lumps that may be removed each day by blowing-off. A really clean boiler is almost unknown unless graphite is used. In certain exceptional cases where the water contains unusually large amounts of solids it may be advisable to use a carefully selected chemical in conjunction with graphite so as to be doubly sure of getting rid of the deposit.

The method of introducing the graphite into the boilers is essentially the same as in feeding compounds. It is introduced into the feed-water line, preferably on the suction side of the feed pump, whence the water carries it to the boilers. The advantage of placing the graphite in the suction line is that it then lubricates the pump valves and packing. The graphite is equally efficacious in lubricating any hot-water meters and in loosening the scale in feed-water heaters, though such benefits are incidental. Anyone can rig up a home-made device for feeding graphite into the pump suction. The apparatus can be made to inject a whole day's supply at one time or may be regulated to give down its charge of graphite slowly throughout the day. The latter method is the best for obvious reasons. If boilers are in a battery, care should be taken that each boiler receives its share of graphite.

The amount of graphite required depends upon the character of the water. Ordinarily from a pint to a quart per day of twelve hours is sufficient for a boiler of



ILL.; PADUCAH & ILLINOIS R.R.
Modjeski, Consulting Engineer.)

up to 250-hp. capacity. Larger boilers need proportionately more. The daily amount may be decreased as soon as the old scale has been brought down. In certain boilers the feed water first enters a mud pan or drum, the purpose of which is to separate from the water as much of the suspended solids as is possible. Naturally the mud drum will collect much of the graphite and prevent it from entering the boiler. As a result the regular prescribed dose of graphite will not be sufficient where the above condition exists, because only part of the graphite is really utilized in the boiler proper. It therefore becomes necessary to feed the graphite slowly throughout the day and to use more than is ordinarily required; otherwise the effect of the graphite on the scale will be of little value.

In comparison with "compounds," boiler graphite costs no more and usually costs less. The expense is overshadowed by the savings from improved steaming conditions. It is rare that graphite will not increase the steaming capacity of boilers at least 25%.

Careful consideration should be given to the grade of graphite best adapted to boiler use. Graphite as found the world over is divided into two general classes: (1) flake or crystalline, and (2) amorphous or powdered. Amorphous graphite is valuable for certain purposes, such as foundry facings or stove polish, where its properties of balling up or sticking together are not objectionable. It is a fact, however, that this form due to the physical characteristics of the particles, to a certain extent forms a mud in the boiler and settles on the plates and tubes. On the other hand, experience has shown that the finely pulverized, thin flake variety will be distributed evenly on the surfaces of the shells and tubes and become more permanently attached to the metal than the amorphous. Both amorphous and flake graphite are employed, but investigation shows that those who manufacture and sell both grades, recommend the finely pulverized flake material in preference to the amorphous, although the margin of profit is approximately the same on both.

A Test Case on the New York Labor Law

The constitutionality of the New York labor law, enacted eight years ago, is to be tested by carrying a case to the Court of Appeals.

The law provides that any public contract shall be void and the contractor shall not be entitled to receive any money thereon, if he requires or permits a laborer, mechanic or workman in his employ to work more than 8 hours in any one calendar day, except in case of "extraordinary emergency caused by fire, flood or danger to life or property"; if he pays less than the prevailing rate of wages for each trade or occupation in the "locality" where the public work is being performed or erected; or if he employs persons who are not citizens of the United States, native or naturalized. The law vests no discretion in public officials, but is mandatory in its penalty of forfeiture of contract for violation of its provisions, in addition to fine and imprisonment.

These drastic provisions of the law have prevented contractors having large contracts involving heavy financial obligations from testing it in the courts, since an adverse decision would mean ruin. The General Contractors' Association has therefore arranged to have test suits brought on three small contracts in Bronx Borough, New York City, where deliberate violations of the law were made to furnish groundwork for the suits.

Mr. Charles R. Crane, Secretary of the General Contractors' Association, has submitted the following statement respecting the evils growing out of the law, as it now stands:

The courts and public officials have adopted a very liberal policy in construing the law in such cases of alleged violation as have come before them. In the eight years that the law has been in operation but one contract has been forfeited for violation of the labor law, and in that instance the contractor's claim for damages was settled out of court by the municipality affected by payment of a substantial sum.

Repeated efforts on the part of the State Labor Department to secure convictions, after finding what appeared to be substantial evidence of violations of the law, have failed. The contractor has always been able either to prove that an "extraordinary emergency' 'existed, or by a "preponderance of evidence" to establish that the complainants were mistaken.

These complaints have been mostly on overtime charges, and many peculiar situations arise which emphasize the defect in the law with regard to the eight-hour provision. It forbids the employer, under heavy penalty, to permit or require an employee to work overtime, but an employee may violate the provision with impunity. It is no uncommon occurrence on large contracts for workmen to make extra time by working in two shifts without the knowledge of the employer. These men are identified by numbers, or brass checks, and it is a simple matter for a laborer to leave his gang a few minutes before quitting time and apply for work at another gang starting on the next shift at a point some distance from his first place of work. With another timekeeper and foreman, it is almost impossible to prevent this, and the explanation of the two checks at the pay window is that one is for a "friend who couldn't come."

Needless to say, no contractor wants a laborer to work sixteen hours at a stretch; he wouldn't permit it, and he certainly wouldn't require it. The laborer wants the money, and he knows he is violating the law, but the only punishment that can be inflicted on him is discharge. In such instances the contractor would not be held responsible and would be in no danger of losing his contract, but the operation of the law is such that he could be put to great inconvenience if the discharged workman filed a complaint that he had worked overtime.

This complaint would serve to stop all payments on the contract until an investigation of the charge had been completed. The contractor would have to employ counsel to conduct his defence, and as the minimum time in which these proceedings can be concluded is about six weeks, in many cases he would have to borrow money to continue his work to replace his withheld payments. His only redress, on being absolved of the charge, is recovery of interest on his payments for the period of their retention.

The provision that no money shall be paid out on a contract pending the investigation of a complaint is a most serious menace to contractors. It is cheaper to buy silence, to submit to blackmail, even when innocent of violations, than to stafid the expense and delay of the investigation. The opportunity to embarrass a contractor is too easy to be overlooked by one having a grudge. Disgruntled competitors have been known to file anonymous complaints, which are just as effective in tying up payments as bona fide affidavits. Affidavits have been filed by dissatisfied workmen, to be withdrawn later if their demands are granted. The special Supreme Court grand jury in New York County which investigated the contract situation on the Catskill Aqueduct and state highways submitted as one of its conclusions:

"That the provisions of the Labor Law, as now administered, with respect to public work, are such as to lend themselves to a use by the unscrupulous for the purpose of extorting tribute from contractors engaged upon such public works."

The clause that causes the most dispute in the Labor law is that requiring that the prevailing rate of wages shall be paid in the "locality" where the work is done Not only is it difficult at times to determine what the prevailing rate is, but the determination of "locality" is open to argument The attorney-general of the state rendered an opinion in a celebrated case that "locality" meant the corporate limits of a city. Yet in the five boroughs of New York City, the rate as established and demanded by labor unions is not

uniform throughout for the same class of work. The highest rates prevail on Manhattan Island.

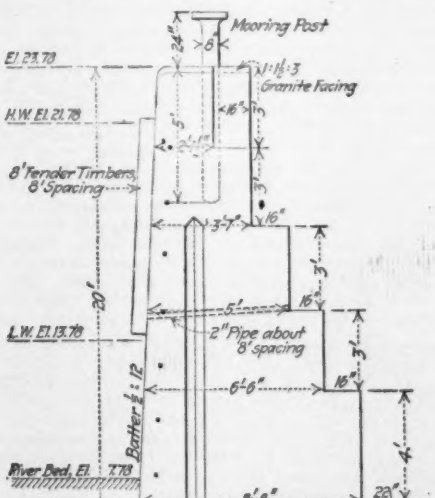
There is rarely a week that one or more complaints alleging that contractors are not paying the prevailing rate are not filed in the Finance Department, and only those persons who are familiar with the construction work now going on in this city can have any idea of the number of cases that are compromised under threat of complaint.

The test case involving the employment of alien laborers is highly important. It is true that this part of the law is honored more in the breach than in the observance, but it offers an opportunity as a "trouble maker" and has been brought up usually during ante-election campaigns to embarrass public officials who are candidates for office. While this law has been declared unconstitutional on two different occasions in this state, these opinions were rendered in trial courts and resulted in the acquittal of the contractor, thus rendering an appeal by the state impossible. The question therefore has not been passed upon by the highest courts, and the law remains on the statute books, fully operative whenever anyone sees fit to call it in use.

The officers of the General Contractors Association are: Franklin Remington, president of the Foundation Co., president; Edward S. Skillin, of Snare & Triest Co., first vice-president; Daniel A. Garber, president of the North Eastern Construction Co., second vice-president; William P. Schmuck, secretary of Degnon Contracting Co., treasurer; C. A. Crane, secretary.

Municipal Harbor Construction in Minneapolis

Improvement of the Mississippi River by a high-lift lock and dam half way between St. Paul and Minneapolis by the Federal government will shortly bring a navigation depth of 7 ft. up to Minneapolis where there has heretofore been only 2 ft. of depth. To develop all the navigation possibilities of this improvement the city of Minneapolis has designed and will shortly start building a municipal wharf. The right bank of the Mississippi immediately below the Washington Ave. bridge will be converted into a wharf by a concrete seawall about 1600 ft. long. There is room for considerable extension downstream and upstream if subsequent developments require. The seawall is to be begun this year.



SECTION OF SEA-WALL FOR MINNEAPOLIS MUNICIPAL WHARF

The river-gorge bluff is fairly close to the bank of the river along most of the length of the seawall, swinging back somewhat at the upper end in the vicinity of the bridge. Along the shore line, however, rock lies close below river bottom. The river bottom as it will be improved by dredging is at Elev. 7.78; rock is struck near this level all along the site of the wall, as test-pits have shown. A height of wall of 20 ft. from foundation to top is counted on as being adequate for all parts of the length. At one point a small run has cut a groove in the rock about 10 ft. below its general level, and this may require slightly deeper foundations.

The wall designed for this work, as shown in the drawing herewith, is a solid concrete gravity wall, with 10-ft. base for the 20-ft. height. It is to be constructed in lengths of 32 ft., separated by contraction joints. The wall will be backfilled level with its top.

The alignment of the wharfage involves a slight angle, about 1000 ft. from the upper end, the lower 600-ft. section swinging outward so as to form a hollow angle with the upper end. This is not likely to have any bearing on the use of the wharf however.

The city engineer plans to erect back of the seawall a warehouse 150 ft. wide, set back from the dock front sufficiently to give room for crane tracks and loading platform. No specific design for this part of the enterprise has been made.

The use of the wharf will require a certain amount of dredging in the river channel abreast of the dock. It is expected that the United States government will do this dredging outside of a line 10 ft. from the face of the seawall, to form a turning basin at the head of navigation.

The plans for the harbor development were worked out by the city engineer's department of the city of Minneapolis, F. W. Cappelen, City Engineer. An appropriation of $75,000 has been made by the city council, and work will start shortly. As in practically all other city work in Minneapolis this year, the day-labor plan of doing the work under direct charge of the city engineer's office will be followed.

NEWS NOTES

A 15-ft. Sewer Trench Caved In when its shoring failed, in Montreal, Que., on July 20, killing one man and injuring another.

A Lifeboat Fell from Its Davits during a boat drill on the White Star steamship "Baltic," at Liverpool, on July 25, killing one man and injuring nine others. The boat fell 40 ft.

The Sunken "Empress of Ireland" has changed her position under the action of strong current, so that her masts are only about 35 ft. below the surface of the St. Lawrence River. Orders were given on July 24 to remove the masts with dynamite.

Burned Ties Wrecked the St. Louis, Iron Mountain & Southern R.R. mail train, one mile south of Diehlstadt, Mo., on July 21. Section men had been burning rubbish on the right of way and the fire had spread to the ties. Six ties were burned from under the rails at the point of the wreck, and a broken rail resulted when the train hit this point. More than 20 persons were injured.

A Broken Railway-Car Truck caused the wreck of a freight train on the East Penn R.R., at Macungie, Penn., on July 22, according to press dispatches. Twenty cars were involved in the derailment of the train, which consisted of 20 steel gondolas. The crippled car was the eleventh back of the locomotive, which went on with the front of the train. None of the train crew was hurt. A runaround was constructed.

A Broken Gas Main filled a manhole, connected to a sewer, with illuminating gas and caused the death of one person, recently, in Toronto, Ont. A main was broken by settling of the earth when the manhole was filled in. The gas passed from the manhole into the sewer. An engineer in the employ of the city was asphyxiated in the new sewer on Argyle St. from this cause. The contractor doing the work stated that the man knew of the presence of the gas four days before his death, and that the gas company had repaired the break a week before the death. Examination of the blood indicated that death resulted from poisoning by carbon-monoxide gas, which gas was present in the sewer in the proportion of eight parts in 1000.

Falling Structural Steel, which was being hoisted by a derrick to the 16th story of a new building being erected in Philadelphia by the Finance Co. of Pennsylvania, blocked traffic for a half-hour by breaking a number of electric wires. The four steel members, weighing about a ton each and about 20 ft. in length, were suspended above the street when the hitch slipped. The beams struck the street, end on, ramming holes a foot deep in the wood-block pavement. They rebounded in the air, falling against the trolley wires, which they bore down with them upon the track. Press reports state that insecure fastenings caused the accident. Just as the workmen were about to guide the four pieces into place one got loose, the others shot after it through the gap left in the chain turns.

High Wind Blew Down a 150-Ton Gantry Crane at the Marion power station of the Public Service Electric Co., of New Jersey, on July 23. The crane, which was parallel to the station, was blown along its track for a distance of about 300 ft. falling over on a yard locomotive. The span of the crane was 150 ft. and the length of the wheelbases 25 ft. Eight wheels on one side were locked by brakes. When the storm broke, the crane was standing at the western end of the track on a coal dock on the Hackensack River. The wind started the crane, which acquired a high rate of speed along the level track, falling at the opposite end of the track. The tracks upon which the locomotive was standing were pushed about two feet into the ground. The damage is estimated at $50,000.

The crane had been in service 10 years. It handled coal from barges to storage piles and from the piles to the hoppers in the buildings.

Nearly a Half-Million Pounds of Dynamite Exploded— The Mindi Powder & Dynamiting Magazine, maintained under the direction of the Deport Quartermaster at Mt. Hope, near the Gatun-Colon wagon road, about five miles from Colon, C. Z., was destroyed by an explosion at 9.20 a.m. July 5. According to the "Canal Record," the magazine contained 435,525 lb. of dynamite and 15,394 lb. of Trojan powder. The day watchman was found dead about 72 paces from the magazine. The family of the night watchman, consisting of himself, wife and seven children, who lived in a small house about 300 yd. south of the magazine, escaped with slight injuries. The wall of the detonator house, about 150 ft. distant, was drawn out, allowing part of the roof to collapse. The shock from the explosion was felt in Mt. Hope, Colon and Gatun, and caused some damage in the way of broken glass, etc. Considerable animal life within a radius of several hundred yards of the explosion was destroyed.

The greater part of the powder and dynamite in the magazine was received on the Isthmus on May 28, since which time it has been inspected at required intervals and, in addition, readings of temperature taken daily. The explosion is thought to have been due to spontaneous combustion.

The magazine was built into a hillside in such a manner that at the back and sides it was surounded by earth. The walls were composed of hollow concrete blocks 12 in. thick, while the roof was of concrete reinforced with old rails. The building was 112x48 ft. in plan and 9 ft. high inside.

Two Steamers and One Large Sailing Vessel Went Ashore on the coast of Cape Breton, N. S., on July 19. All three struck within a few hours of each other, in a fog accompanied by a heavy landward swell. The first accident reported was that of the American 3-masted schooner, "Harold C. Beecher," bound from New York for Sidney, N. S., with a cargo of sand. The crew landed safely. A few hours later the Cuban steamer, "Cienfugos," of 1139 tons, and laden with pitch pine, bound for Montreal, went ashore. The crew of 20 were landed in a lifeboat despatched from the East Light life station. The second steamer to get into difficulty was the Norwegian "Ragna," bound from Chester, Penn., for St. Anne's, C. B., for cargo. Her crew of 20 reached shore safely. The "Ragna" has a registered tonnage of 1062 and is six years old. It was known on July 20 that none of the three vessels could be saved.

Other dispatches received at Halifax stated that the Austrian steamer "Irene" was ashore at Woody point, Baul[illegible] and that the American schooner "Clarence C. Venner" crash[illegible] on the southwest ledges off Cape Sable. The crew was sa[illegible] The "Clarence C. Venner" was bound for Halifax from N[illegible] York with 1300 tons of anthracite.

On July 19 the Italian bark "King Malcolm," bound fr[illegible] Portland, Me., to Richibucto, N. B., in ballast, was lost abo[illegible] seven miles from Yarmouth, N. S.

On July 18, the 3-masted schooner "Jennie A. Pickles," of Annapolis Royal, N. S., went ashore in a thick fog at Chan[illegible] Harbor, in the Bay of Fundy. She was on her way fr[illegible] Bermuda to St. Johns, N. B., with a cargo of molasses.

June Floods Destroy Hundreds of Iowa Highway Bridges— May and June floods cost Iowa, it is estimated, $200,000 or more in damaged and destroyed bridges. What they cost in washed out grades and minor damage to highways, there is no possible way at the present time to even approximately guess. The estimate on bridge damage and loss is made by the field engineers of the State Highway Commission, who have been called to many counties by reason of the emergency work. The failures were due chiefly to two reasons, foundations insufficient and not properly placed reinforcing. In some cases of failure, the concrete material was of the best, but lack both of sufficient and properly placed reinforcing developed weaknesses that under ordinary circumstances would never have been detected. The washing out of creek and ditch bottoms, in some cases 9 or 10 ft. deep, disclosed many cases of the most woeful disregard of all engineering practices in the building of foundations. Concrete bridges were disclosed that had been resting on a thin film of black muck overlying quicksand. Many bridges went out for the simple reason that the wing walls were not securely tied by reinforcing to the abutment. This, with lack of depth in the foundation, allowed the flood water to sweep away the foundation of the wing wall, the lack of reinforcing allowed the wing to break from the abutment and topple, and with the fill melted away, the wrecking of the bridge was easy. Every concrete bridge wrecked had been built by bridge contractors on their own plans without other than their own inspection, or by day labor by the counties themselves, sometimes on no plans at all. There is no record so far of a properly designed and constructed concrete bridge being destroyed.—[July "Service Bulletin" of the Iowa Highway Commission.]

Explorations for Two Dam Sites are being carried on by the Aluminum Company of America on the Little Tennessee River, near the Tennessee-North Carolina boundary. One of the proposed sites is just above Alcoa, Tenn., and the other is nine miles above at the mouth of the Cheowah River, in North Carolina. The river runs through a deep gorge in both places. It is planned to develop 100,000 hp. to be transmitted to a reduction plant at Maryville.

Highway Work in Pennsylvania has begun in earnest. The release of the funds derived from the taxation and registration of automobiles, as noted in these columns of July 2, p. 52, amounts to about $1,100,000, which can now be drawn upon by the State Highway Commissioner. According to newspaper reports no attempt will be made at new construction until repair and maintenance work has been accomplished. It is said between 3000 and 4000 men are already at work on the roads of the state.

Mesquite Wood Blocks have become so scarce that the city of San Antonio, Tex., has abandoned the plan of paving 250,000 sq.yd. of streets with this wood. The contract was let in April for over 43,000 sq.yd. of mesquite block paving, but the contractor has been unable to furnish the blocks. Mesquite is a very hard, tough, dense wood, almost indestructible, and has been used for the sills of adobe houses in the Southwest for very many years. In many respects it makes an ideal material for paving.

Repetition of the Pittsburgh Sewer Explosion of Nov. 25, 1913 (see our columns, Jan. 1, 1914, p. 22) occurred on Tuesday morning, July 28, 1914. It was a very small explosion, compared with the unprecedentedly violent and extensive one of last November. The 33rd St. trunk sewer near its outfall end was the seat of the explosion. The Lower Union Mill, of the Carnegie Steel Co. was slightly damaged. Repair and reconstruction work on the sewer has been going on continuously for the past seven months, the explosion of last November having destroyed a length of one mile of the sewer. This reconstruction is practically completed; some workman had just left the sewer at the point of the accident when the explosion occurred.

A New Railway Bridge Over the Ohio at Sciotoville is to be built by the Chesapeake & Ohio Northern Ry. Co., a subsidiary company of the Chesapeake & Ohio Ry. The former company has asked permission of the War Department to erect a

bridge across the Ohio River a few hundred feet above the mouth of the Little Scioto River, at Sciotoville, Scioto County, Ohio. The bridge is to consist of two 750-ft. cantilever spans, with a south approach of 14 and a north approach of 11 deck plate-girder spans of 80 ft. each. Bids on the bridge have been called for.

The Road Congress at San Francisco—A general road congress, to be held at San Francisco during 1915, conducted jointly by the American Highway Association and the American Road Builders Association is being planned for. The plan was approved at a special meeting of the directors of the American Road Builders Association on July 17. The management of the joint meeting would be in the hands of a committee of five made up of two members from each association and a fifth member to be selected by these four. The meeting will probably be held immediately following the International Engineering Congress.

Six Large Contracts on the New York Rapid Transit system's extensions have recently been executed by the Public Service Commission. The aggregate amount of the six contracts is over $21,000,000. Two of these contracts were for the construction of the two East River tunnels, contract for which has been let to the Flinn-O'Rourke Co., an organization made up of the firms of Booth & Flinn, and the O'Rourke Engineering & Construction Co. A part of the Eastern Parkway subway in Brooklyn was awarded to the Cranford Co., the Canal St. subway in Manhattan to the Underpinning & Foundation Co., part of the Lexington Ave. subway in Manhattan to the Rapid Transit Subway Construction Co., and the Broadway subway from 26th to 38th St., to the United States Realty & Improvement Co.

The Record for Placing Concrete Masonry in Kensico Dam in the month of May, 1914, noted in "Engineering News," June 4, 1914, p. 1276, was surpassed in the month of June. The record is as follows:

Number of working days (8 hr.)	26½ days
Total masonry placed in dam, all kinds	76,970 cu.yd.
Average amount of masonry per day	2,905 "
Best day's work	3,314 "

Three gravity mixers, each taking 8-bag batches (nominal 2-yd mixers), were used for making the concrete. The average daily output per mixer, with no deductions for delays, was 669.7 cu.yd., or 83.7 cu.yd. per hour measured in place. Three-quarters of all concrete was mixed by two mixers located on the downstream side of the dam. The average daily output of each of these two mixers, with no deductions for delays, was 783.3 cu.yd., or 97.9 cu.yd. per hour, measured in place. The best day's output of one mixer was 1142.8 cu. yd., or 142.8 cu.yd. per hour, measured in place.

A Deep Water Diving Suit was tested in Long Island Sound off Stamford, Conn., on July 26, in a depth of 212 ft., which is probably a record for deep sea diving. The suit is in effect a metal casing, looking curiously like a suit of Chinese armor and jointed at the elbow, knee, hip and shoulder to allow movement of the diver. One arm is provided with a searchlight and the other with a grappling hook. The casing completely envelopes the diver and is of sufficient strength to resist pressures at least as great as that experienced in the test. According to the owner's reports, "no attempt is made to make the suit airtight, so that all of the water that works in through the joints is kept out by an automatic pump." Air for the diver is supplied through a pipe to the helmet, which pipe also contains a telephone wire.

The diver making the record was Arthur Gentsch, a well known deep-sea diver. The demonstration was under the direction of the Submarine Diving and Recovering Co.

An Electricity Rate of 18c. per kw.-hr. was recently protested to the Massachusetts Board of Gas and Electric Light Commissioners by the city of Northampton. An order of the Board governing the rates of the company in question, the Northampton Electric Light Co., has just been published. The company charged 18c. per kw.-hr. gross, for commercial supply, with a discount of 4c. for prompt payment, for the first 15 kw.-hr. For all in excess of 15 kw.-hr., the price was 11c. with 1c. discount for prompt payment. Net prices for power ranged from 6.7 to 3.0 per kw.-hr.; there were various special-service rates also. The prices for street lighting were $90 per year for all-night arcs, and $70 for midnight arcs with $5.56 extra per lamp for early morning operation. Several hundred 50-watt tungsten street lamps were used at $28 and $19 (all night and midnight). By special arrangement 12 street arches, carrying 208 50-watt tungstens were charged at $1200.

The company made a general reduction to 12c. maximum net commercial rate, $85 and $71 for arcs, and $24 and $18 for the 50-watt tungstens. The city asked for a general rate of 7c. per kw.-hr. for lighting, 5c. for street lighting and 2c. for power.

The Board reported that since 1888 (when the company began to report to the Board) the concern had failed to pay dividends in but two years. In only seven of the remaining had less than 6% been paid. In 1902, it began paying 8%, and in 1906, 10%. The annual dividends in the last five years had averaged over 20% and a surplus above interest, expenses and dividends of over $19,000 per year had been secured. The book value of plant and assets was $294,295, against which there were $206,878 in stocks, bonds and debts.

The Board fixed a rate of 9c. per kw.-hr. maximum net general rate; for arc lamps, $82 and $68; for tungsten street lamps, $17 and $13.50. The Board refused to touch other rates, holding that they were made from motives of expediency to obtain competitive business which could not otherwise be secured.

PERSONALS

Mr. F. L. Beal has been appointed Valuation Engineer of the Louisiana & Arkansas Ry., with offices at Stamps, Ark., under the direction of the General Manager.

Mr. W. A. Ebbert, formerly with the Pekin Wagon Co., Pekin, Ill., is now in charge of the dump wagon department of the Kentucky Wagon Manufacturing Co., Louisville, Ky.

Mr. Fitz J. Lewis, M. Am. Soc. M. E., Supervisor of the Pacific locks, Panama Canal, resigned, effective July 14, and sailed for New York July 16. He entered the Canal Service on Apr. 11, 1906, as a machinist in the shops of the Mechanical Division at Empire.

Mr. S. N. Mills, former Chief Clerk of the Block Signal and Train Control Board, Interstate Commerce Commission, has been appointed Inspector, and has been assigned to the work of investigating and testing signal and train control systems and other safety devices.

Mr. John L. Mohun, formerly with the mechanical department of the Pennsylvania R.R., is now Assistant to the Consulting Engineer of the Union Pacific R.R., the Oregon Short Line R.R. and the Oregon-Washington R.R. & Navigation Co., with headquarters at New York City.

Col. Henry G. Prout, M. Am. Soc. C. E., has resigned as President of the Union Switch & Signal Co., of Pittsburgh, Penn., to which office he was elected in April, 1914. A sketch of Col. Prout's career as a soldier, engineer, editor and business man appeared in these columns of Apr. 2.

Mr. E. L. Powers, Editor and owner of "Good Roads," New York City, and Secretary of the American Road Builders Association, was quite seriously injured in an automobile accident near Jamestown, N. Y., July 23. It is stated that he is now recovering and is resting comfortably in a Buffalo, N. Y., hospital.

Maj. G. A. Young, Corps of Engineers, U. S. A., has been appointed Professor of practical military engineering at the United States Military Academy at West Point, according to act of Congress, Apr. 15, 1914, which makes the head of that department at the Academy, hitherto known as Instructor, a Professor with the rank of Lieutenant-Colonel, U. S. A.

Mr. Charles D. Pollock, M. Am. Soc. C. E., has resigned as Chief Engineer of Highways, San Antonio, Tex., and has opened an office at 21 Park Row, New York City, as Consulting Engineer, specializing in paving. Before going to San Antonio, Mr. Pollock was Chief Engineer of Paving of Havana, Cuba. Previous to that he was with the Bureau of Highways of the Borough of Manhattan, New York City.

Mr. Henry R. Towne, Past-President, Am. Soc. M. E., President of the Yale & Towne Manufacturing Co., New York City, has been elected a Class B Director of the Federal Reserve Bank of New York, as a representative of commercial interests. The Class A Directors must be bankers. Mr. Towne has long been active in the affairs of the New York Merchants' Association and was President of that organization from 1908 until June, 1913.

Mr. Harold A. Petterson, Assoc. M. Am. Soc., C. E., Civil and Hydraulic Engineer, San Francisco, Calif., formerly Field Superintendent for the Union Oil Co. of California, left San Francisco, Calif., July 11, for China, where he has been appointed Professor of structural and hydraulic engineering at the University of Pei Yang at Tien Tsin. Mr. Petterson is a graduate of the University of California and was connected with the engineering staff of the Los Angeles aqueduct.

Mr. J. B. Lippincott, M. Am. Soc. C. E., of Los Angeles, Calif., has been selected by the Public Utilities Commission, of Denver, Colo., as its Consulting Engineer in connection with the proposed plan of constructing a new water-supply system for the city and county of Denver. Mr. Lippincott

was Chief Assistant Engineer throughout the construction of the Los Angeles aqueduct. The problem of providing a complete water-supply system for Denver is very similar to the Los Angeles problem. Mr. Lippincott will undertake the preliminary studies at once.

Mr. Ralph Adams Cram, F. Am. Inst. Arch., of Boston, Mass., has been appointed Senior Professor of architecture at the Massachusetts Institute of Technology, succeeding Mr. James Knox Taylor, F. Am. Inst., Arch., resigned. Mr. Cram will continue in active practice and will devote only a part of his time to the Institute. Prof. William H. Lawrence, a graduate of the Institute and for many years Professor of architectural engineering, has been appointed Chairman of the department of architecture and will be responsible for the administrative routine.

Mr. Max W. Enderlein, formerly a Reclamation Service Engineer, of Hanford, Calif., has been appointed as one of the assessors to determine the values of land along the Sacramento and San Joaquin Rivers, California, which will be taxable under the new "Flood Control" Bill. About 2,000,000 acres of river land, located in 14 counties fall within the scope of the contemplated assessment. The project has already been approved by California, and the rivers and harbors bill, now pending in the national congress, if passed, will donate $6,000,000 to this undertaking.

The permanent staff for the operation and maintenance of the Panama Canal, which is soon to be opened for commercial traffic, is as follows:

Col. H. F. Hodges, M. Am. Soc. C. E., Corps of Engineers, U. S. A., is Engineer of Maintenance; A. B. Nichols, Office Engineer; Capt. W. H. Rose, Assoc. M. Am. Soc. C. E., Corps of Engineers, U. S. A., Electrical Engineer, Electrical Division, Balboa; Geo. M. Wells, M. Am. Soc. C. E., Resident Engineer, Division of Municipal Engineering, Gatun. H. H. Rousseau, M. Am. Soc. C. E., Civil Engineer, U. S. N., is Engineer of Terminal Construction, Culebra; F. H. Cooke, Assoc. M. Am. Soc. C. E., Civil Engineer, U. S. N., Designing Engineer; T. B. Mönniche, M. Am. Soc. C. E., Engineer of Docks, Cristobal; J. A. Walker, Superintendent, Balboa; W. G. Thompson, Superintendent, Cristobal. W. G. Comber is Resident Engineer, Division of Dredging, Paraiso. D. C. Nutting, Jr., Constructor, U. S. N., is Superintendent, Mechanical Division, Balboa. Capt. Hugh Rodman, U. S. N., is Superintendent of Transportation, Division of Canal Transportation, Ancon; Com. D. E. Dismukes, U. S. N., Captain of the Port, Cristobal; Lieut.-Com. H. V. Butler, U. S. N., Captain of the Port, Balboa; W. F. Beyer, Assistant Engineer, Lighthouse Construction, Ancon. First-Lieut. A. H. Acher, U. S. A., is Assistant Engineer, Division of Fortifications, Culebra.

Mr. Edward Schildhauer, M. Am. Soc. C. E., has resigned as Electrical and Mechanical Engineer, Panama Canal, and has returned with his family to the United States. He entered the Canal Service as Electrical and Mechanical Engineer on Nov. 15, 1906. During his period of service, he has been occupied principally with the designing, manufacturing, and installation, under the direction of Col. H. F. Hodges, formerly Assistant Chief Engineer, and since Apr. 1, Engineer of Maintenance, of the machinery and electrical equipment for the operation of the canal locks, spillways, and power plants, including the new hydroelectric station at Gatun, the substations at Gatun, Cristobal, Miraflores, and Balboa, and with the new transisthmian transmission line. Mr. Schildhauer invented and patented the bull wheel, which operates the lock gates. He also invented and patented the system of electric towing locomotives for handling ships in the locks. Another novel device, manufactured from designs prepared in his office, is the system of remote control, by which every part of the lock operating machinery is manipulated from a central station or control house, and where, with the aid of a reproduction of the lock and its operating features in miniature, every movement is registered before the eyes of the operator. Mr. Schildhauer was born at New Holstein, Wis., entered the University of Wisconsin at the age of 21, and was graduated 1897, with the degree of Bachelor of Science in electrical engineering. The Master's degree was conferred in 1911. He was engaged in electric railway construction until 1898, when he entered the employ of the Chicago Edison Co., and was with this company at the time of his appointment to the Canal Service. He is president of the Panama Section of the American Institute of Electrical Engineers, chairman of the Canal Zone Chapter of the American National Red Cross Society and a member of various societies in the United States, including the American Institute of Electrical Engineers, the American Society of Civil Engineers, the American Society of Mechanical Engineers, the National Electric Light Association, and the American Electro-Chemical Society. He will make his headquarters at the Engineer's Club, New York City. He was given a farewell reception by the Tivoli Club on Saturday evening, July 11, and by the University Club on Wednesday evening, July 15. [From the "Canal Record" July 15.]

OBITUARY

William Yearian, a young civil and mining engineer of Boulder, Colo., was drowned while swimming in a reservoir near Boulder, on July 10. He was formerly in mining work in Mexico. He was 26 years old.

W. S. Becker, former Assistant to the President of the Norfolk & Western Ry., and for many years General Superintendent of the Western division of the railway, died July 19 at his home in Bluefield, S. C.

Marc Spencer, a young civil engineering graduate of the University of North Carolina, was killed July 21, by a fall, while employed on construction work for the Southern Aluminium Co., at Salisbury, N. C. He was 24 years old.

Horace Crosby, M. Am. Soc. C. E., of New Rochelle, N. Y., died July 24. He was one of the founders of the city's public library and was at one time President of the Board of Education. He was one of the oldest members of the American Society of Civil Engineers, having been elected in 1869.

David O. Fowler, President of the Uvalde Asphalt Paving Co., New York City, died at his summer home in Lawrence, Long Island, N. Y., July 22, aged 54 years. He was a great grandson of Admiral Compte de Grasse, the French naval ally of Washington in the Yorktown campaign which ended the Revolutionary War.

Horace H. Moses, an old-time civil engineer and in the early 80's Superintendent of Sewers of Boston, Mass., died at his home in Roxbury, Mass., July 21. He was born at Exeter, N. H., in 1831. In 1848 he went to Boston and entered the office of a civil engineer. During his later life he devoted much time to landscape architecture.

William Perry, a locomotive engineer whose experience dated back to the earliest days of American railroading, died July 23, at his home in Rochester, N. Y. He was born in Wales in 1821 and emigrated to America with his parents in 1828, when he was 7 years old. The family settled in Utica, N. Y., where young Perry attended school until he was 18 years old, when he went to Rochester and entered the employ of the Auburn & Rochester R.R., an early link in the New York Central & Hudson River R.R. He continued in active service from 1839 until 1880.

ENGINEERING SOCIETIES

COMING MEETINGS

AMERICAN ASSOCIATION OF RAILROAD SUPERINTENDENTS.
Aug. 20-21. Annual meeting in New York City. Secy., E. H. Harman, Room 101, Union Station, St. Louis, Mo.

AMERICAN PEAT SOCIETY.
Aug. 20-22. Annual meeting at Duluth, Minn. Secy., Julius Bordollo, 17 Battery Place, New York City.

AMERICAN BOILER MANUFACTURERS' ASSOCIATION.
Sept. 1-4. Annual convention in New York City. Secy., J. D. Farasey, E. 37th St., and Erie Ry., Cleveland, Ohio.

AMERICAN MINE SAFETY ASSOCIATION.
Sept. 7-10. Annual meeting in New York City. Secy., H M. Wilson, Bureau of Mines, Pittsburgh, Penn.

NATIONAL ASSOCIATION OF PORT AUTHORITIES.
Sept. 8-10. Annual convention in Baltimore, Md. Secy., Wm. Joshua Barney, 29 Broadway, New York City.

ROADMASTERS AND MAINTENANCE OF WAY ASSOCIATION OF AMERICA.
Sept. 8-11. Annual meeting at Chicago, Ill. Secy., L. C. Ryan, Sterling, Ill.

MASTER CAR AND LOCOMOTIVE PAINTERS' ASSOCIATION.
Sept. 8-11. Annual convention in Nashville, Tenn. Secy., A. P. Dane, Reading, Mass.

NEW ENGLAND WATER-WORKS ASSOCIATION.
Sept. 9-11. Annual convention in Boston, Mass. Secy., Willard Kent, Narragansett Pier, R. I.

INTERNATIONAL ASSOCIATION OF MUNICIPAL ELECTRICIANS.
Sept. 15-18. Convention in Atlantic City, N. J. Secy., Clayton W. Pike, Electrical Bureau, Philadelphia, Penn.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 21-25. Annual convention in Cleveland, Ohio. Secy., Joseph D. Israel, 29 W. 39th St., New York City.

RAILWAY SIGNAL ASSOCIATION.
Sept. 22-24. Annual convention in Bluff Point, N. Y. Secy., C. C. Rosenberg, Times Building, Bethlehem, Penn.